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LOCK APPROACH CANAL SURGE AND TOW SQUAT AT LOCK AND DAM 17 ARKANSAS RIVER PROJECT

Mathematical Model Investigation

by

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The upstream approach canal to Choteau Lock is 150 ft (bottom) wide and has a nominal depth of 9 ft. Navigation tows with 8- to 8-1/2-ft draft have encountered navigation problems in the canal. These problems include several incidents of grounding on the canal bottom while under way. The lock filling surge is also considered a problem. This investigation was initiated to help the Tulsa District determine the most effective means of resolving navigation problems.

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An unsteady flow mathematical model was used to compute lock filling surges in the canal for the present and a variety of canal geometries. Squat effects in the canal were computed for several canal depths and widths, using a simplified model developed for steady and transient squat effects.

Results showed that canal widening provides greater surge reduction than canal deepening for a given increase in cross-sectional area. Steady squat model computations indicate that a 12-ft depth is the minimum necessary to eliminate grounding. Transient squat computations at canal transitions (called supersquat effects) showed that a 200-ft-wide (bottom width) canal with a minimum 13-ft depth below el 511 would reduce grounding problems with minimum excavation costs. A 12-ft-deep by 300-ft-wide canal (i.e. without transitions) was recommended if economically justifiable. This canal size would allow increased tow speeds in the canal, eliminate supersquat altogether, and improve canal navigation conditions.

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PREFACE

The study described in this report was conducted during the period July 1977 to March 1979 for the U. S. Army Engineer District, Tulsa, by the U. S. Army Engineer Waterways Experiment Station under the general supervision of Messrs. H. B. Simmons, Chief of the Hydraulics Laboratory, and M. B. Boyd, Chief of the Hydraulic Analysis Division. A preliminary report presenting the results, conclusions, and recommendations from the investigation was presented to the Tulsa District in March 1979.

The unsteady flow mathematical model calculations and tow squat model development and application were accomplished by Mr. Carl Huval. Mr. Huval also prepared this report for publication. Special thanks should go to Dr. Billy Johnson and Mr. Paul Senter for help in model application and data preparation. The writer appreciates the help of Dr. Garbis Keulegan and Mr. M. B. Boyd and Mr. Frank A. Herrmann, Jr., for their careful review and suggestions on this report.

Commanders and Directors of WES during the conduct of this study and the preparation and publication of this report were COL John L. Cannon, CE, and COL Nelson P. Conover, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
cubic feet per second	0.02831685	cubic metres per second
cubic yards	0.7645549	cubic centimetres
feet	0.3048	metres
feet per second	0.3048	metres per second
feet per second per second	0.3048	metres per second per second
miles (U. S. statute)	1.609344	kilometres
square feet	0.09290304	square metres
tons (2000 lb, mass)	907.185	kilograms

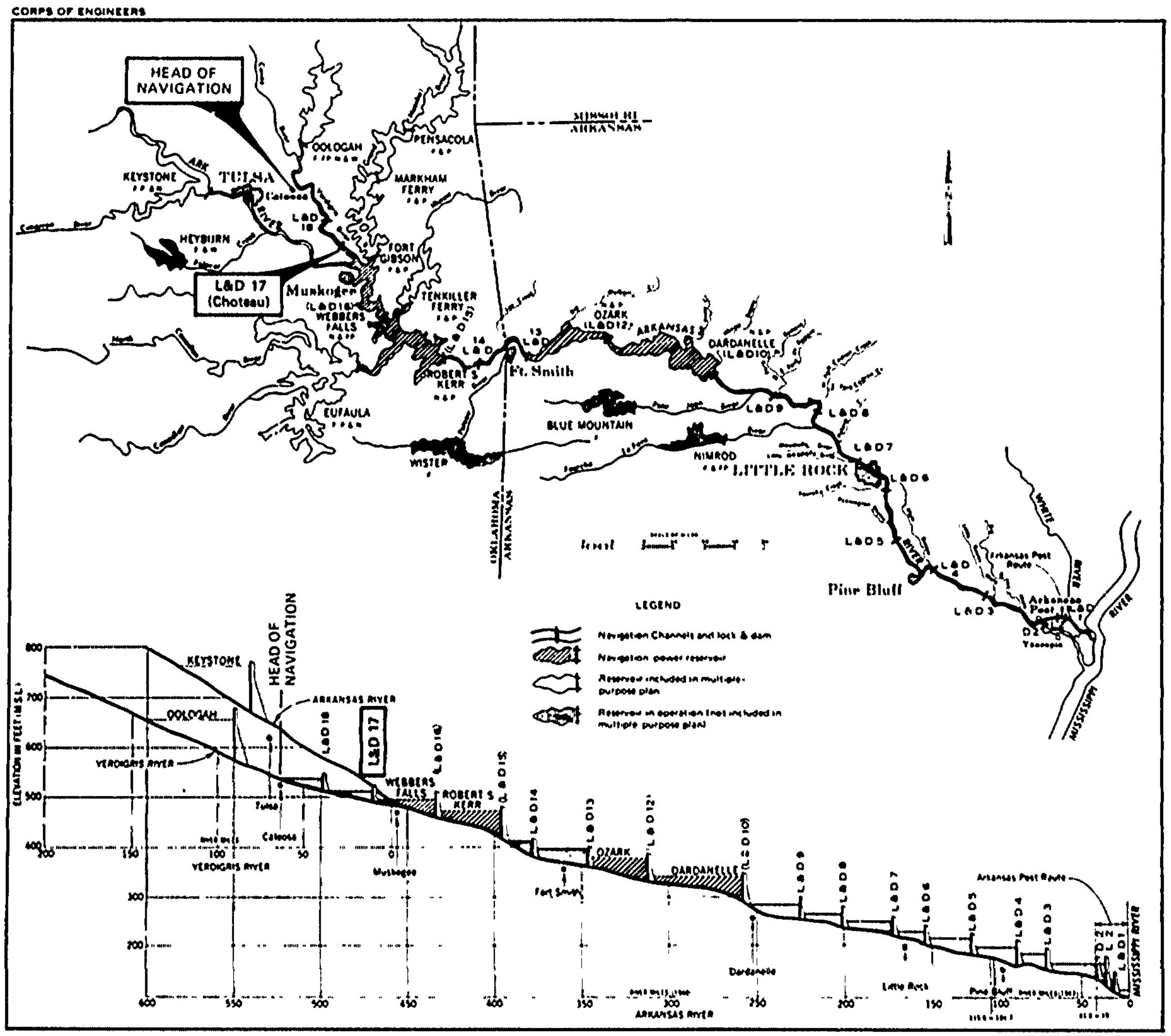

Figure 1. Location map

LOCK APPROACH CANAL SURGE AND TOW SQUAT AT

LOCK AND DAM 17, ARKANSAS RIVER PROJECT

Mathematical Model Investigation

PART I: INTRODUCTION

Project Description

1. Lock and Dam 17 (also known as Choteau Lock and Dam) is located on the Verdigris River a few miles downstream of the head of navigation of the Arkansas River project at Catoosa, Okla. (Figure 1). The navigation features of the project provide for a 9-ft-deep* channel by a system of locks and dams on the Verdigris and Arkansas Rivers. The lock chamber is 110 by 600 ft long. Minimum channel width of 150 ft was dredged for the Verdigris reach of the project.

2. After considering several plans, Dam 17 was built on the Verdigris River at mile 9.8, while the lock was located in a canal about 3400 ft east of the Verdigris River channel. Figure 2 presents the layout of the pertinent features of the project. The normal upper pool at L&D 17 is at el 511.0** and the normal lower pool established by Dam 16 (Webbers Falls) is at el 490.0. The lock has a normal lift of 21 ft and a maximum lift of 24 ft. The dam and spillway as built consisted of three 60- by 27-ft tainter gates with the spillway crest at el 485.0 and an earth embankment extending from the spillway to the lock. Details of the canal entrance and spillway location were developed with the aid of a physical model study at the U. S. Army Engineer Waterways Experiment Station (WES) (Franco, Glover, and Melton 1970).

3. The upstream approach canal to Lock 17 is nominally 150 ft wide and 9 ft deep for a distance of about 5560 ft as shown in Figure 2.

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

** All elevations (el) cited herein are in feet referred to mean sea level (msl).

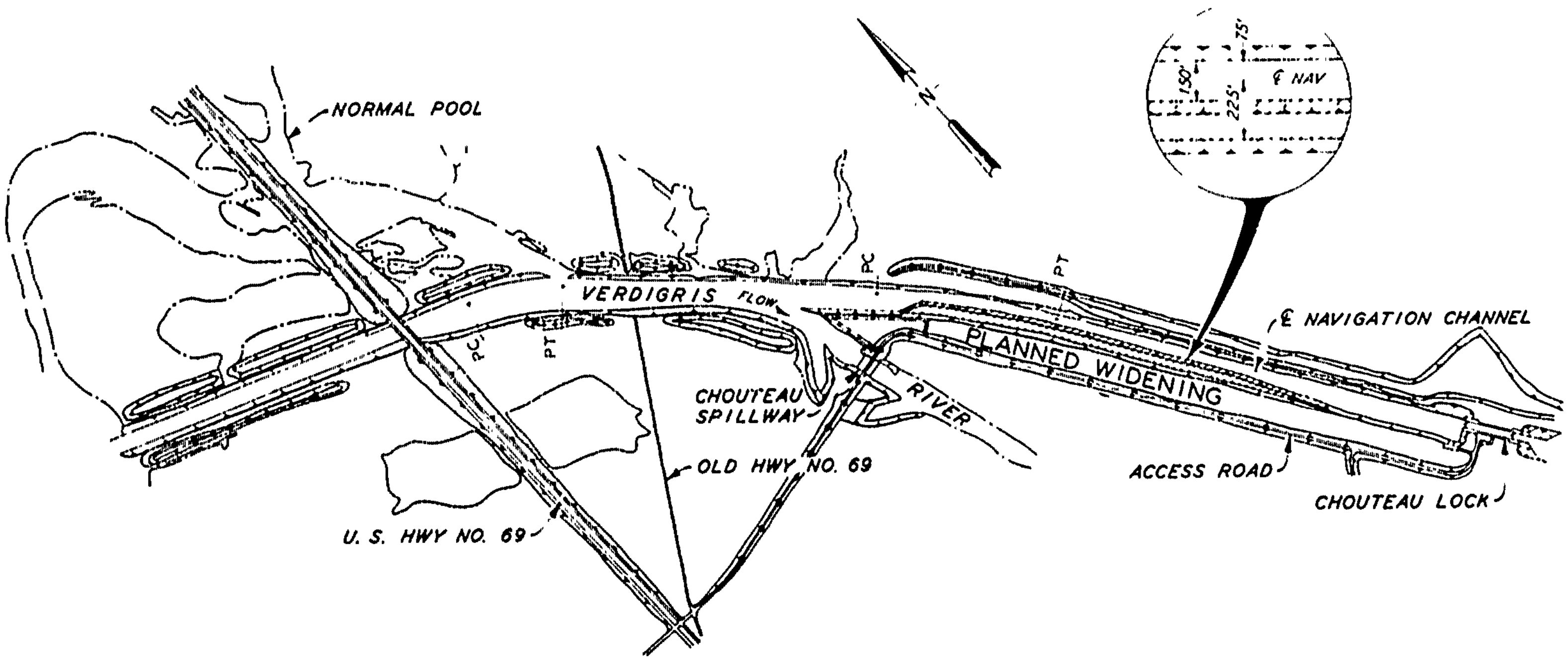

Figure 2. Project layout

Details of the canal and river dimensions are given in Table 1. The canal includes two wide reaches to aid navigation and reduce surge effects at both ends of the 150-ft canal. A 1416-ft-long reach with 300-ft bottom width is provided at the canal entrance from the Verdigris River. A 521-ft-long reach by 300-ft bottom width was also excavated near the upstream lock approach. The canal includes two transitions from the nominal dimensions of 150-ft bottom width to 300-ft bottom width. The upstream 1:2 transition contracts the canal as tows travel downbound and the downstream 1:10 transition contracts the canal as tows travel upbound. The 150-ft-wide canal conforms to the authorizing document minimum width for the project. The design, however, included provision for future widening as warranted by navigation traffic.

4. Most of the towboat traffic operating on the Verdigris River is in the 2000-hp class but regular runs are being made by a few boats having up to 4200 hp. Tows longer than 600 ft experience difficulty negotiating the Verdigris channel bends. Most of the loaded tows on the Verdigris are 102 to 108 ft wide by 7- to 8-1/2-ft draft and slightly under 600 ft long, which is the maximum size that can be accommodated in the lock without breaking tows. Figure 3 shows a typical canal cross

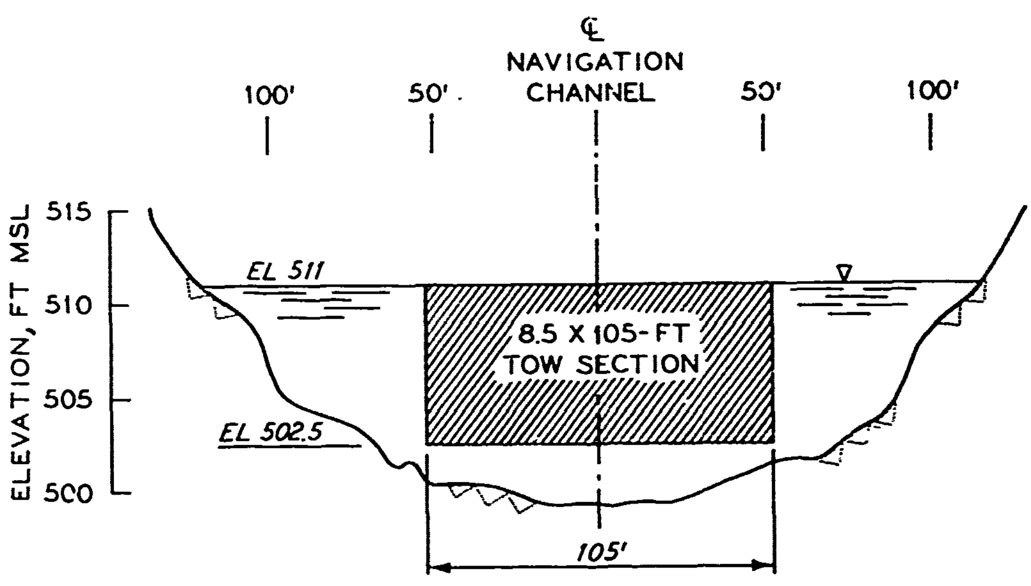

Figure 3. Tow in canal

section with an 8-1/2-ft draft by 105-ft beam tow in the canal. There is only about 1/2- to 1-ft clearance, depending on position along the canal, on each edge of the canal as shown in Table 1. The tow cross section is a major fraction of the total canal section.

Navigation Problems

5. Navigation problem reports from towboat operators have been received by the lockmaster on a number of occasions. The large frontal area of most of the tows in comparison with the canal cross-sectional area has been observed to cause a towboat to squat as much as 1-1/2 to 2 ft below its static floating position, depending on tow size and speed. Information from the field indicates that tow grounding problems occur mainly while maneuvering through the canal transitions and when accelerating in the canal. However, most of the larger towboats can ground on the bottom going steady ahead in the narrow canal reach. Problems with grounding on the bottom by loaded tows occur both upbound and downbound. The upbound tows report problems at a point which is about 600 ft downstream of the beginning of the 1:10 transition (Figure 2). Downbound tows ground at a location about 200 ft downstream of the beginning of the 1:2 transition.

6. The squat problem can be further aggravated by filling the lock chamber while a downbound tow is in the approach canal. If possible, the lock is filled well before downbound tows reach the approach canal to minimize the effects from the lock surge. As traffic increases, however, this operational constraint could cause considerable delays. Field tests have shown that filling of Lock 17 can cause as much as 1.3 ft of drawdown at a point 1 mile upstream from the lock. Squat or the lock filling surge can cause grounding of the tow, especially when the pool elevation is low and the depth is at the minimum of 9 ft. Combinations of lock surge and tow squat conditions could cause grounding of a severity that could lead to rupturing a barge or breaking up a tow.

7. Due to the difficulties from lock filling surge and tow squat, the approach canal to Lock 17 has had insufficient depth to accommodate

fully loaded tows. The U. S. Army Engineer District, Tulsa, thus requested authority to widen the upstream approach channel at Lock 17 from 150 to 300 ft and to increase the depth from a nominal 9 to 12 ft. Factual data were not sufficient to verify this plan to be the most economical and effective method to relieve the problem. It was suggested that simply widening the channel from 150 to 300 ft may not eliminate grounding, particularly at times when the channel depth in only 9 ft. Thus, increasing depths from 9 to 14 ft may be much more cost-effective for an equivalent amount of excavation. It was desirable that the design of the canal remedial plans should consider the quantities of materials involved for each of several alternatives to get a relative idee of the costs for each plan.

8. WES was requested to conduct a numerical model investigation of the lock surge and towboat squat in the Lock 17 approach canal. The numerical model study was conducted to determine the most effective means for resolving the navigation problem and to minimize the towboat squat and lock surges from filling operations. The purpose of the study was to assist the Tulsa District in determining the minimum canal width and depth that would provide adequate navigation conditions.

Scope of Study

9. The canal surge problem was modeled mathematically using an available unsteady flow model (Johnson 1974). This model had been used to study several open-channel flow problems including surge conditions in the Illinois Waterway resulting from lock filling and emptying operations. The region modeled for this study includes the 8500-ft-long canal upstream from the lock and a 1210-ft-long reach of the Verdigris River to a point just upstream of the old Highway 69. Figure 4 shows a sketch of the channel system that was modeled. Lock surge field tests were conducted in September 1976 and March 1978 and these data were used for calibration.

10. Squat effects in the canal were computed in separate calculations assuming the towboat is moving at a steady speed ahead in the

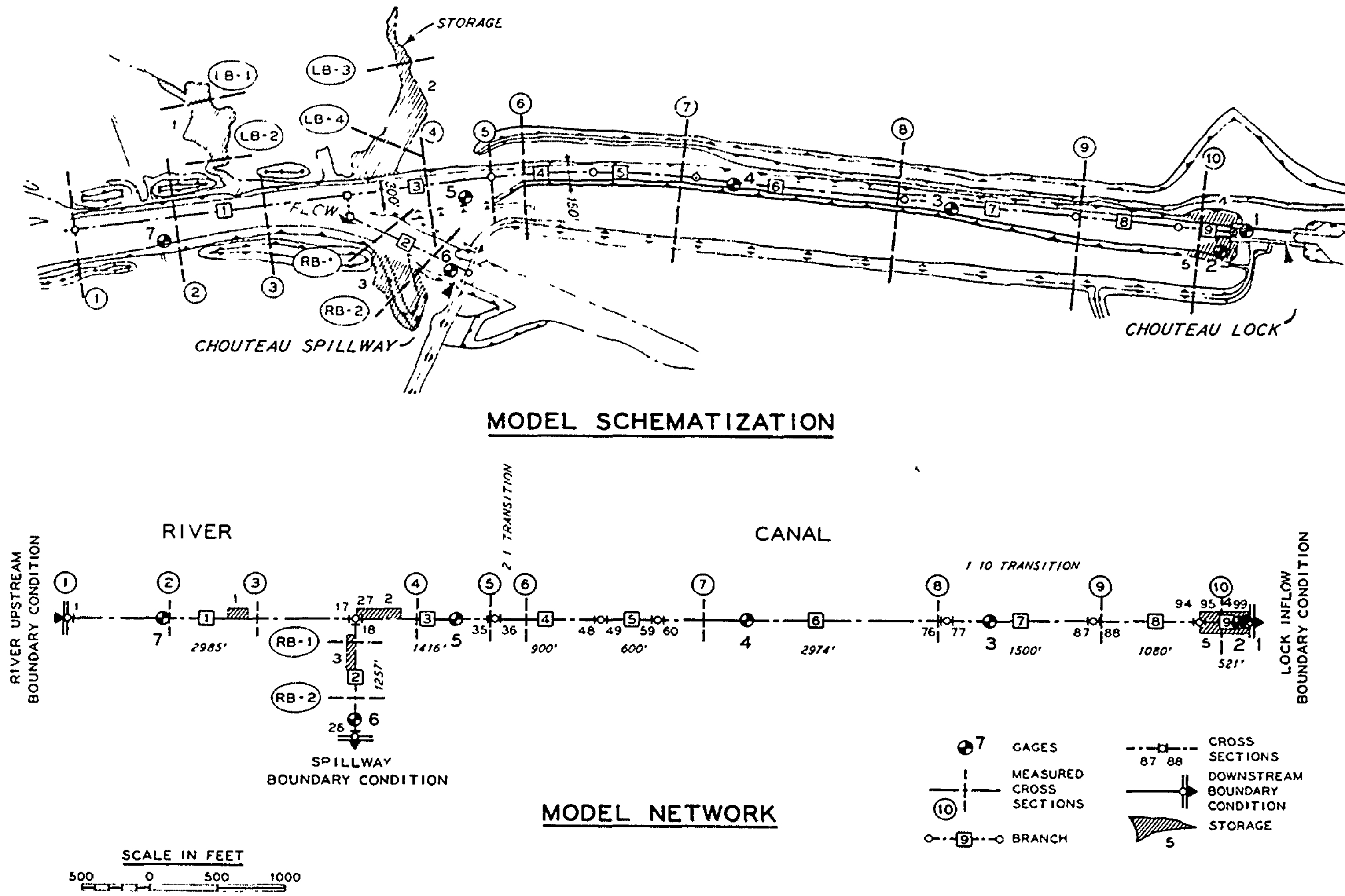

Figure 4. Channel system

canal. The steady squat effects were modeled using the approach outlined by Schijf (1953). The model study included comparative surge and squat calculations of canals with the following depths and widths:

Width ft	Depth ft
150	9-12-14-15
200	9-12-14-15
250	9-12-14-15
300	9-12-14-15

These cross sections are shown in Figure 5. The design tow size was chosen as 8-1/2-ft draft and 105-ft width, which gives about 893-sq-ft cross-sectional area.

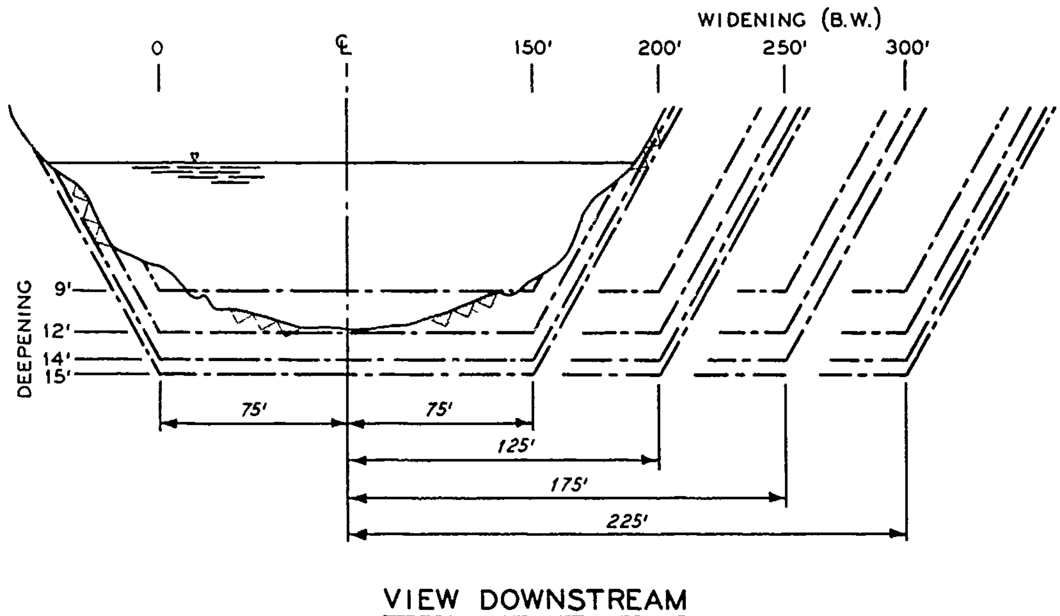

Figure 5. Canal cross section

PART II: MODEL DESCRIPTIONS

Lock Surge Model

11. The numerical model used to determine lock filling surge heights along the lock canal is a model called SOCHMJ (Simulation of Open Channel Hydraulics in Multi-Junction Systems) (Johnson 1974). The development of the computer program called SOCHMJ has been documented in the cited report and has been in general use by the Math Modeling Group for flood routing, tidal computations, and channel surges. The equations solved by the model are based on unsteady flow and are derived in books on open-channel flow (Henderson 1966). The resulting equations of motion can be written as:

$$\text{Continuity:} \quad \frac{\partial h}{\partial t} + \frac{1}{W}\frac{\partial(AV)}{\partial x} - \frac{q}{W} = 0 \qquad (1)$$

$$\text{Momentum:} \quad \frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial h}{\partial x} + \frac{gn^2V|V|}{2.21R^{4/3}} = 0 \qquad (2)$$

where

- h = water-surface elevation above mean sea level*
- $\partial/\partial t$ = rate of change with respect to time
- W = top width of water surface
- A = cross-sectional flow area
- V = mean flow velocity
- $\partial/\partial x$ = rate of change with respect to distance
- q = lateral inflow per unit distance along the channel per unit time
- g = acceleration due to gravity
- n = Manning's resistance coefficient
- R = hydraulic radius

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* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix A).

These equations are often referred to as the equations of St. Venant. Since analytical solutions don't exist, numerical techniques such as finite differences must be employed. The centered explicit finite difference scheme developed by Stoker has been utilized in SOCHMJ.

12. Since an explicit finite difference computation scheme is utilized, the stability criterion

$$\left(V + \sqrt{g\frac{A}{W}}\right)\frac{\Delta t}{\Delta x} \leq 1 - \frac{gn^2|V|\Delta t}{2.21R^{4/3}}$$

must be satisfied by the time and spatial steps, i.e., Δt and Δx , to ensure a stable solution of the difference equations.

13. Data required for the operation of SOCHMJ are read from computer cards. The first data card contains basic information such as the total number of net points in the system, the total number of junctions and branches, and the time-step employed in the computations. For the application to the lock approach canal these values were 99, 7, 9, and 3 sec, respectively. Figure 4 shows the model network and Tables 2 and 3 give the schematization data. The three outer branch ends are considered to be boundary conditions and not junctions. The second data group contains information about each of the nine modeled branches. Such information consists of the numbers of the first and last net points of each branch, the type of boundary conditions prescribed at the various boundaries, and the size of the spatial step to be employed for each branch. The third data group contains information about the junctions in the system being modeled such as the numbers of the branches joining each junction. The next major data group consists of the tables of geometric data. A table of top width, flow area (hydraulic radius)$^{2/3}$, and Manning's n (all as functions of elevation) must be input at each net point in the system. These tables were developed from cross-sectional plots as in Figure 3 furnished by the Tulsa District for canal and river cross sections as shown in Figure 4. The tables consisted of nine elevation data value entries to describe the channel geometry. The Manning's n used was 0.025 for all the computations.

14. The fifth data group specifies initial values of the elevation and discharge at all grid points on the first two time lines. Initial conditions were set for zero flow and 511.3-ft elevation throughout the model. Sensitivity tests indicated these initial conditions have a very minor effect on canal surges due to initial steady riverflow of 7000 cfs as compared with the no-flow condition. The final major data group required by SOCHMJ consists of the time-dependent boundary conditions which must be prescribed at each open boundary. At such a boundary, elevations, discharges, or a rating curve may be prescribed as the boundary condition. For the study described herein, a time-discharge plot of lock filling furnished by the Tulsa District was used on the downstream end of the model as shown in Figure 6. Adjustments required in the lock filling curve are discussed later in the section on model calibration. Several trial computations were conducted to set the boundary conditions on the upper ends of the model. Based on these tests, a zero inflow boundary condition in the upper end of the river and an elevation of 511.3 at the spillway were used.

15. Tabular computed output can be obtained from SOCHMJ at specified stations along the modeled reach as well as at all or specific computational net points. At such net points, output in the form of elevations, velocities, and discharges plus geometric data, if desired, is provided. The output time interval is variable and can be selected in the input data.

Tow Squat Model

Steady squat

16. The squat of a ship or tow in a canal can be determined based on a one-dimensional model apparently first formulated in 1901 by Thiele (Van de Kaa 1978) but greatly augmented by Schijf (1953). If the tow is moving steady ahead in a canal, there is acceleration of the water particle velocity past the sides and bottom of the tow and, by Bernoulli's equation, a consequent lowering of the water level. A definition sketch of the model is shown in Figure 7. By the principle of flow

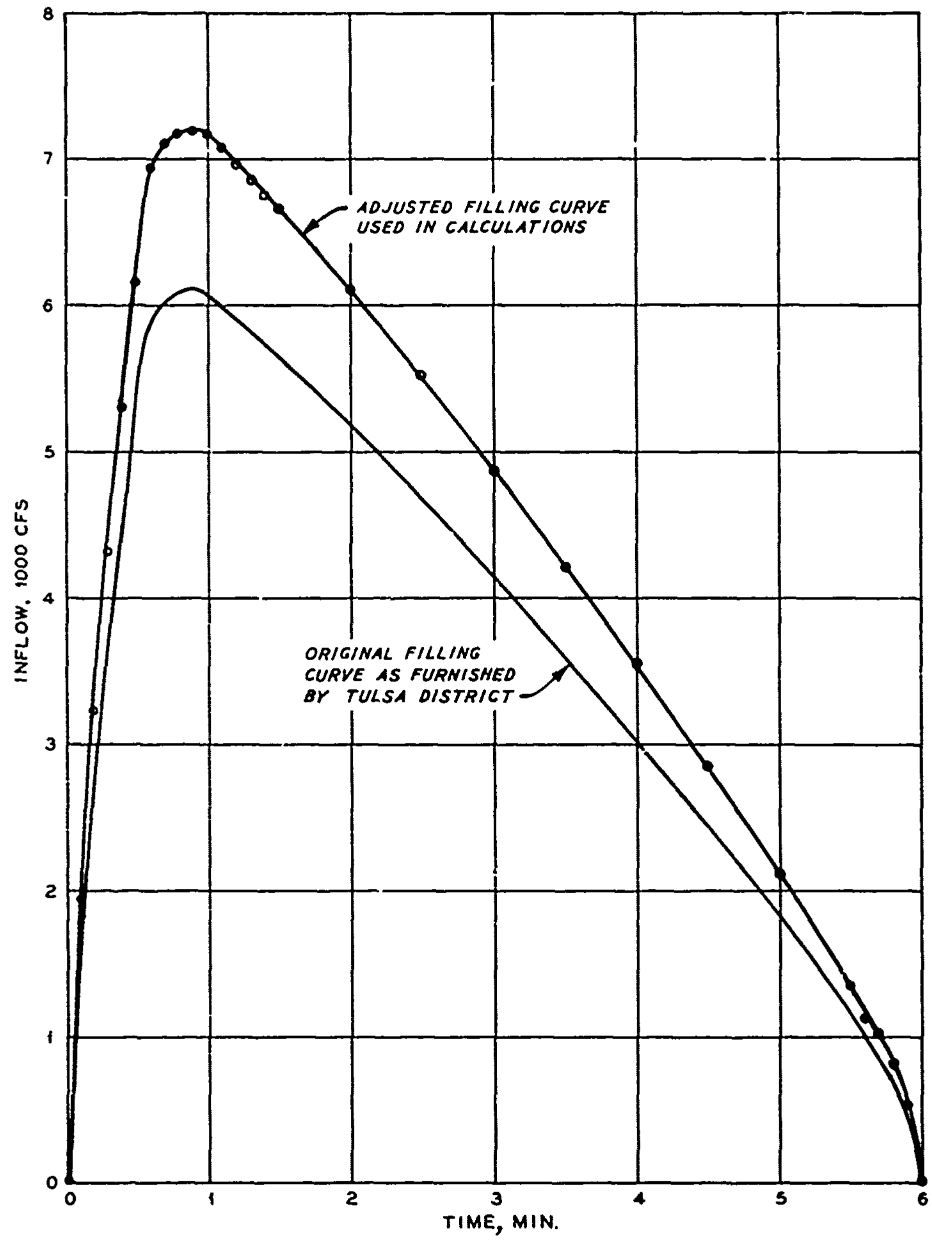

Figure 6. Lock chamber filling curves

superposition, the unsteady flow is made steady with respect to an observer moving with the tow. The tow is stationary with an approach velocity equal to the towboat's driving speed. The resulting equations are

$$V_tA = (V_t + u)(A - a - Wz) \tag{3}$$

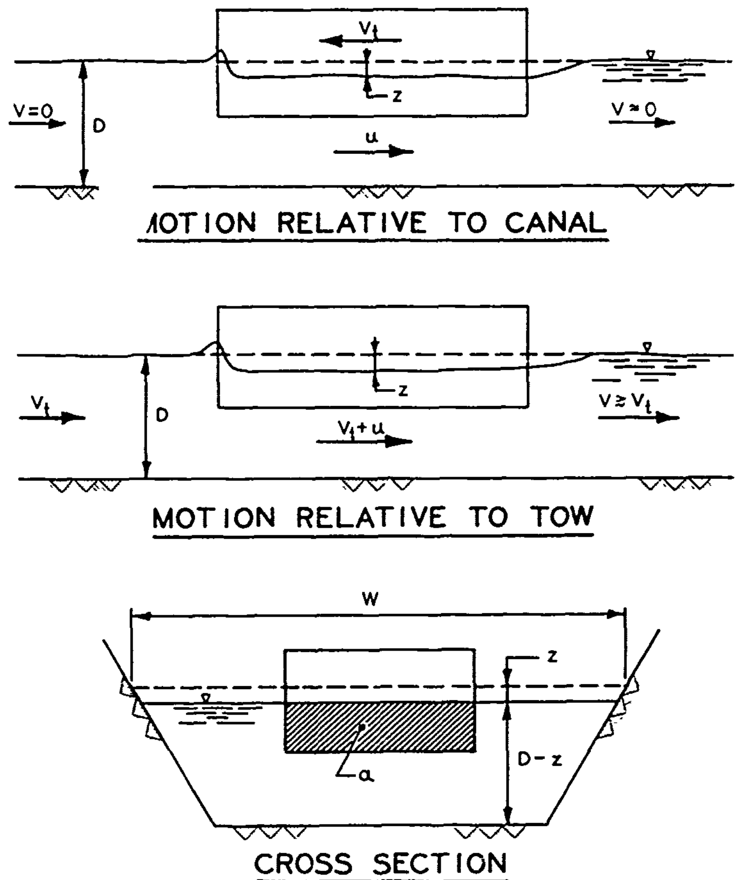

Figure 7. Squat model definition sketch

and

$$\frac{V_t^2}{2g} = \frac{\left(V_t + u\right)^2}{2g} - z \qquad (4)$$

where

V_t = steady ahead tow speed

u = return velocity

z = vertical tow squat

W = canal top width at the undisturbed water level

A = canal area at the undisturbed water level

a = statically submerged tow area

The assumption is made in the above that the squat and the canal width variation is small and the decrease in canal area can be approximated by a rectangle. The average channel depth $\overline{D} = A/W$ can be used to describe

the water depth for most practical canal cross sections using the approximation.

17. The equations can be reduced to the following

$$z = \frac{u^2 + 2uV_t}{2g} \tag{5}$$

$$u = V_t \left(\frac{a + Wz}{A - a - Wz} \right) \tag{6}$$

The tow squat for a given canal, tow size, and speed can be calculated from the above by trial for the squat model. An initial estimate of u is found by ignoring z in the second equation to obtain

$$u_1 \approx V_t \left(\frac{1}{n - 1} \right)$$

where

$$n = \frac{A}{a}$$

The first estimate of the return velocity u_1 is used in Equation 5 to obtain

$$z_1 \approx \frac{V_t^2}{2g} \left[\frac{2n - 1}{(n - 1)^2} \right]$$

The improved value z_1 is used for a refined estimate of the return velocity in Equation 6

$$u_2 \approx V_t \left(\frac{1 + \frac{nz_1}{\bar{D}}}{n - 1 - \frac{nz_1}{\bar{D}}} \right)$$

and the squat is further improved by using

$$z_2 = \frac{1}{2g}\left(u_2^2 + 2u_2 V_t\right)$$

The iteration for calculation of squat and return velocity is thus seen to take the following forms

$$z_{k+1} = \frac{V_t^2}{2g}\left[\left(\frac{1 + \frac{nz_k}{\overline{D}}}{n - 1 - \frac{nz_k}{\overline{D}}}\right)^2 + 2\left(\frac{1 + \frac{nz_k}{\overline{D}}}{n - 1 - \frac{nz_k}{\overline{D}}}\right)\right] \quad (7)$$

$$u_{k+1} = V_t\left(\frac{1 + \frac{nz_{k+1}}{\overline{D}}}{n - 1 - \frac{nz_{k+1}}{\overline{D}}}\right) \quad (8)$$

The error tolerance was set to terminate the iterations by

$$\frac{|z_{k+1} - z_k|}{z_{k+1}} \leq 10^{-4}$$

An HP65 program was developed to calculate the squat and return velocity using the calculation procedure outlined above and is given in Table 4.

Schijf limiting speed

18. One of the most important results from the Schijf theory is that at certain tow speeds, critical flow (in the open-channel flow sense) will occur adjacent to the tow. As a tow approaches this Schijf limiting speed, tow squat increases to the highest level, waves and surges create navigation and canal bank hazards, and tow resistance becomes very high. The limiting speed is thus an important upper speed

parameter and can be calculated using the following explicit formula given by Balanin (1977)

$$\frac{V_\ell}{\sqrt{g\bar{D}}} = \sqrt{8 \cos^3 \left[\frac{\pi}{3} + \frac{\cos^{-1}}{3} \left(1 - \frac{1}{n}\right)\right]} \tag{9}$$

19. When tow speeds approach V_ℓ (say $V_t \approx 0.95\ V_\ell$), the iteration for squat calculations presented in Table 4 will not converge. The following formula gives the maximum squat at V_ℓ :

$$\frac{z_\ell}{\bar{D}} = \cos \left[\frac{\pi}{3} + \frac{\cos^{-1}}{3} \left(1 - \frac{1}{n}\right)\right] - 4 \cos^3 \left[\frac{\pi}{3} + \frac{\cos^{-1}}{3} \left(1 - \frac{1}{n}\right)\right] \tag{10}$$

The return velocity at V_ℓ can also be calculated and is given by the formula

$$\frac{u_\ell}{\sqrt{g\bar{D}}} = \sqrt{2 \cos \left[\frac{\pi}{3} + \frac{\cos^{-1}}{3} \left(1 - \frac{1}{n}\right)\right]} - \sqrt{8 \cos^3 \left[\frac{\pi}{3} + \frac{\cos^{-1}}{3} \left(1 - \frac{1}{n}\right)\right]} \tag{11}$$

Computations were made using these equations by means of the HP65 program shown in Table 4.

Tow Squat at Canal Transitions

20. As a tow maneuvers downbound through the lock approach canal, two transitions are encountered. The first is a contraction from 300 to 150 ft wide and farther downstream an expansion from 150 to 300 ft wide. The canal narrows 150 ft in a longitudinal distance of 300 ft at the first transition (1:2). Several cases have been reported of tows grounding on the canal bottom while maneuvering through this transition.

Downbound tows have little difficulty in the canal expansion farther downstream. On the other hand, upbound tows have reported incidents of tows "setting on the bottom" near the upstream end of the 1:10 transition (150-ft expansion in a longitudinal distance of 1500 ft). The following paragraphs present the methodology that was used for analyzing the "supersquat" phenomenon when tows are driving through canal transitions. The method of analysis is based on previous work by Hooft (1969) and Marchal (1977) as well as the review report by Kolkman (1978).

21. As tows proceed in the 300-ft-wide canal reaches toward either canal transition, the speed will likely approach the Schijf limiting velocity V_ℓ . A value of 90 percent of V_ℓ was the maximum self-propelled tow attainable speed achieved during recent studies by the Delft Hydraulics Laboratory (Van de Kaa 1978). Tows usually proceed at much lower speeds, but 90 percent was used for computations. Computations to be presented later show that $0.9V_\ell$ in the present wide (300 ft) canal will be higher than the limiting velocity in the narrow (150 ft wide) canal reach. The squat on the tow will thus increase in the narrowed canal by a substantial amount, and the velocity will exceed the maximum or limiting velocity in the narrowed canal. Experiments and theory both indicate considerable surge action at and near the limiting velocity. Additional flow cannot pass the tow in the narrowed canal section and the resulting flow "choking" will cause a positive surge to be generated upstream of the tow. This surge height will be a function of the difference between the tow speed in the wide canal and the limiting velocity in the narrow canal reach. Figure 8 helps to clarify the phenomenon and gives the definition of terms.

22. Using the flow superposition principle as previously given for the steady ahead squat (paragraph 16), the tow is made stationary in the transition with an approach velocity V_t allowing a quasi-steady flow analysis. The velocity of the tow is given the magnitude of 90 percent of the limiting velocity in the wide canal reach (1)

$$\left(V_t\right)_{(1)} = 0.9\left(V_\ell\right)_{(1)}$$

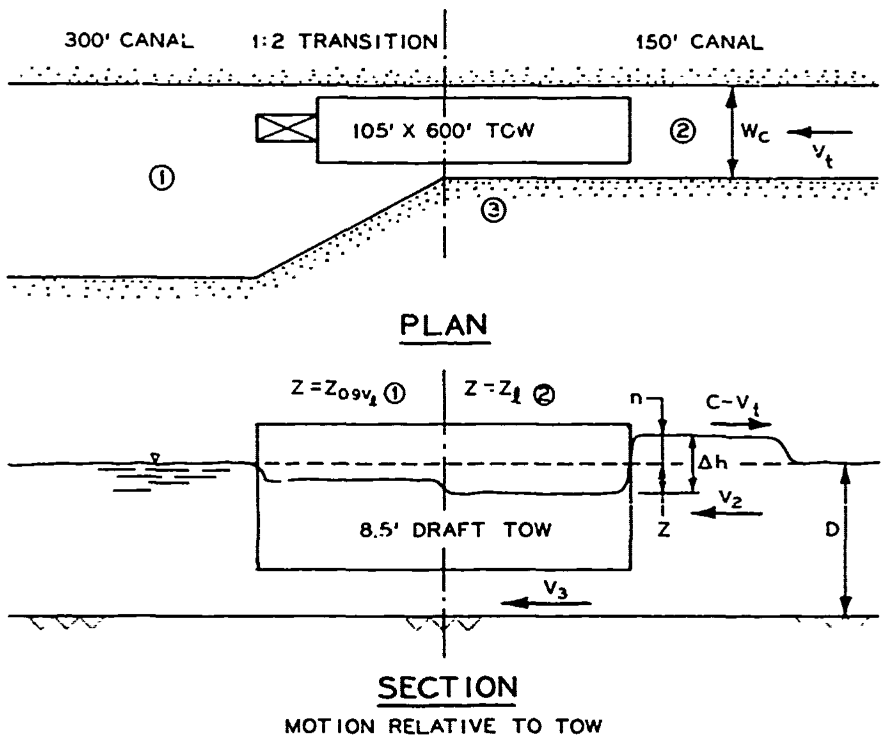

Figure 8. Tow in canal transition

If this velocity is greater than the limiting velocity in the narrow canal (2), flow past the tow will be controlled by the limiting velocity $(V_\ell)_{(2)}$ and a positive surge of amplitude η will be generated at the bow of the tow. If $(V_t)_{(1)}$ is equal to the limiting velocity at (2), η becomes zero. If $(V_t)_{(1)}$ is less than the limiting velocity at (2), then the normal sublimit squat relation applies. As the tow encounters the increased resistance from the positive surge η, the tow speed will become less the $(V_\ell)_{(2)}$, and approach the value

$$(V_t)_{(2)} \rightarrow 0.9(V_\ell)_{(2)}$$

23. The magnitude of the maximum squat z can be calculated for the given $(V_\ell)_{(2)}$ using the steady-speed squat model previously

described. Using the values of $\left(V_\ell\right)_{②}$ from the squat model and the tow speed in the wide canal, the positive surge η can be calculated using the surge continuity equation

$$\left[\left(V_t\right)_{①} - \left(V_\ell\right)_{②}\right](A + W\eta) = c\eta W \qquad (12)$$

where A and W are taken at ②.

$$\frac{\eta W\sqrt{g\overline{D}}}{A + W\eta} = \left(V_t\right)_{①} - \left(V_\ell\right)_{②}$$

$$\frac{\eta\sqrt{g\overline{D}}}{\overline{D} + \eta} = \left(V_t\right)_{①} - \left(V_\ell\right)_{②}$$

If η is a small quantity with respect to $\overline{D}$, the equation can be simplified to

$$\eta = \frac{\overline{D}}{\sqrt{g\overline{D}}}\left[\left(V_t\right)_{①} - \left(V_\ell\right)_{②}\right] \qquad (13)$$

24. It is evident from the discussion presented that the transient tow supersquat at the 1:2 transition near the upstream end of the canal is greater than the steady ahead squat. The bow of the tow will usually react hydrostatically to the sum of $z_\ell + \eta$ (defined as Δh), thus causing the tow stern to sink by that amount relative to the undisturbed water level. This Δh is the supersquat at the stern of the tow.

25. Tows grounding and other navigational problems have also occurred at the 1:10 transition farther downstream. It appears, therefore, that increasing the length of transition may not eliminate the problem. Due to the 600-ft length of many tows using the canal, it may be expected that superlimiting velocity will occur at the bow of the design tow

proceeding in the 1:10 transition. In general, it would be necessary to make the canal transitions very gradual so that area differences between the bow and stern of the tow cannot lead to superlimiting velocities at the bow. The present 1:10 transition, for example, leads to an area difference of about 600 sq ft between tow bow to stern. This is about one-third the present 150-ft-wide canal cross-sectional area indicating that the transition section will cause a large impact on squat. Transition lengths of three or more times the present length of 1500 ft would be necessary to decrease area changes below 10 percent of the present canal area.

26. The analysis presented assumes the tow is proceeding at constant speed in the 300-ft-wide canal reach without any change in engine setting. It is probable that some pilots would attempt to regain speed through the transition by increasing engine rpm. This would further complicate the problem at a transition and would lead to an increased tendency for tow grounding.

PART III: LOCK SURGE MODEL APPLICATIONS

Calibration

27. Lock surge field data were obtained in July and September 1976 and surge tests were repeated in March 1978. The July 1976 tests were made at a riverflow of about 24,000 cfs and the September 1976 tests at 7200 cfs. Four staff gages were located at 1/2-mile intervals in the lock approach canal with two more on the upstream river channel. Location of these gages is shown in Figure 4; gage 1 was located in the lock chamber. The 1976 tests involved a series of three lock filling and emptying operations. The test operations were conducted so as to maximize the canal surges. Measurements were made at 1-min intervals for about an hour. Only the negative (lock filling) surges were of interest in this study, since the problem was one of lack of adequate depth for navigation.

28. The 1978 test was conducted at 7600-cfs riverflow and consisted of one lock filling operation with data obtained at 1-min intervals for 1-1/2 hr. The upper pool elevation was 511.30 at the beginning of the test. Plots of the three data sets indicated only minor experimental differences between the three test series during the first 15 min of testing. The September 1976 and March 1978 data were used for model calibration.

29. Figures 9, 10, and 11 present water-surface elevation time-history field data plotted at three of the four canal staff gage locations (gages 3, 4, and 5, respectively). The numerical model computations using the lock filling curve originally furnished by the Tulsa District (Figure 6) showed a less severe negative surge than the field data. The maximum difference at gage 3, for example, was about 0.2 ft. When the lock filling curve in Figure 6 was integrated for the 6-min lock filling time and compared with the known volume of the lock (21 × 110 × 670 ft), a 17.84 percent difference was noted. As a result, the original lock filling curve was adjusted by increasing all ordinates by 17.84 percent. The revised model computations after lock filling

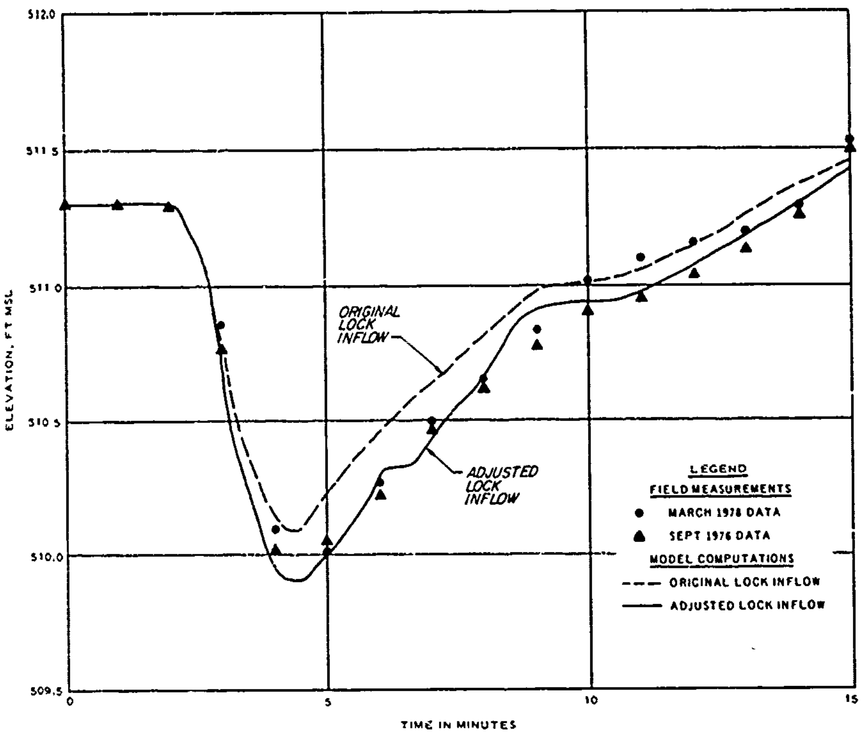

Figure 9. Model calibration, gage 3

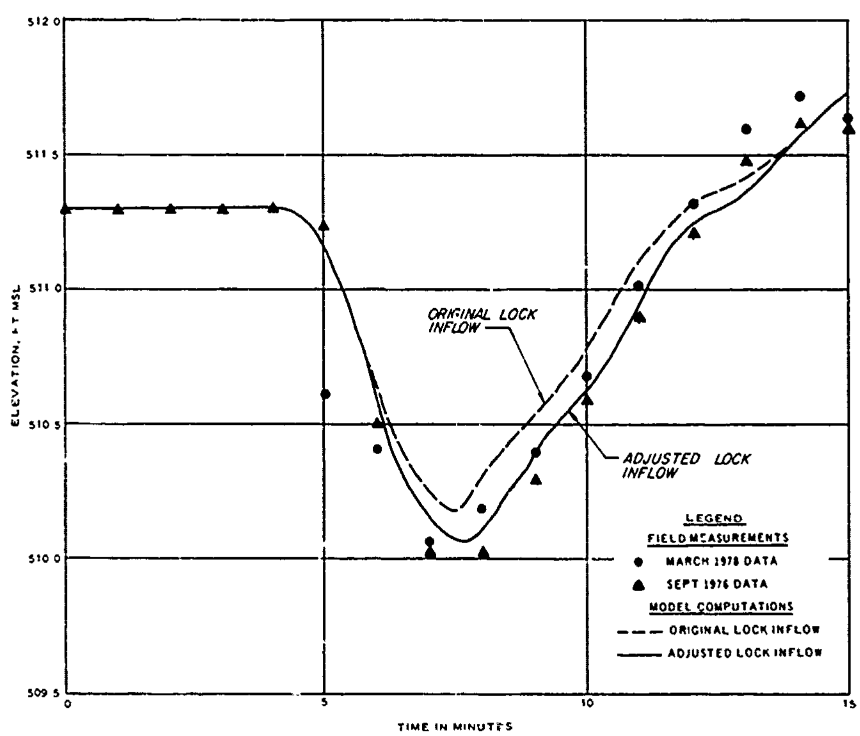

Figure 10. Model calibration, gage 4

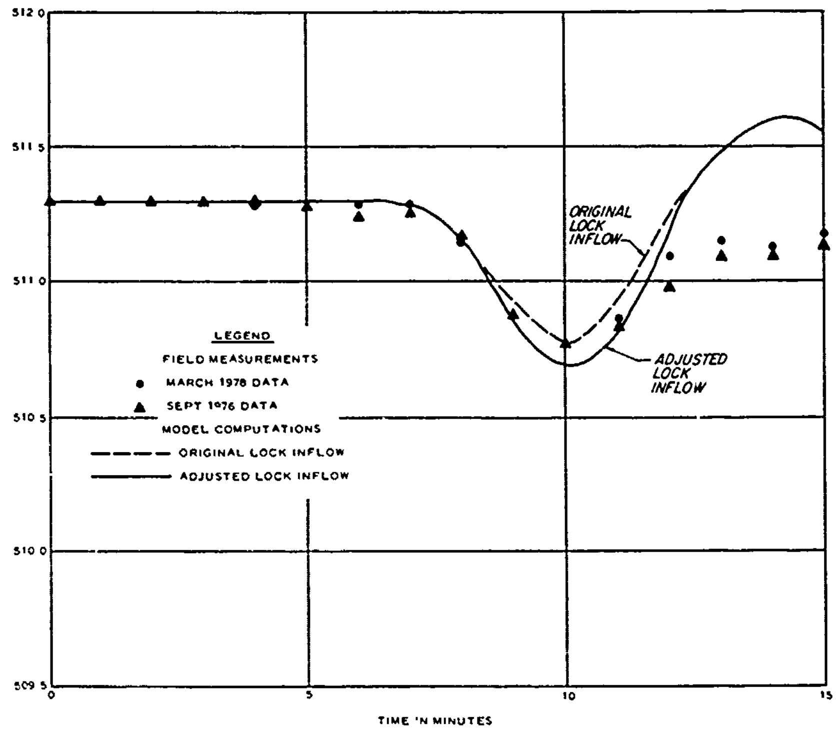

Figure 11. Model calibration, gage 5

rate adjustment are also shown in Figures 9, 10, and 11, and they indicate much better agreement with the field data. Differences between field data and computations at gage 5 after about 11-1/2 min are attributed to nonrealistic spillway boundary conditions. The primary canal surge problem centers around the maximum, initial negative surge. For the purpose of this study, only computation results during the initial wave propagation through the canal for times less than 11-1/2 min were used for further analysis.

Existing Conditions

Surge in existing canal

30. Computation results for the existing canal can be used to ascertain the extent of the lock filling surge problem in the present canal. Several plots are presented and discussed in the following

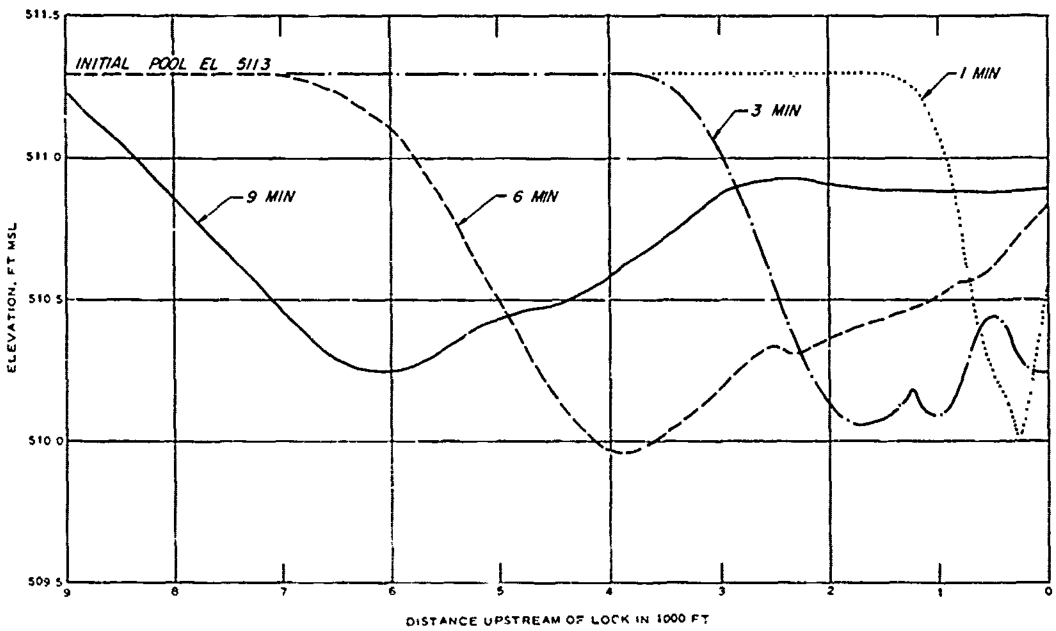

Figure 12. Surge profiles without tow

paragraphs to portray the present problem and lay the basis for analyzing the effects of canal modification alternative plans. Figure 12 presents surge profiles along the approach canal at 1, 3, 6, and 9 min after start of lock filling. The negative surge front tends to flatten during propagation upstream. The tail of the front can be quite complex, due to the variation of lock filling rate and surge interaction with the complex geometry in about the first 1500 ft upstream from the lock (branches 8 and 9). The surge magnitude tends to decrease during upstream propagation. The speed of surge propagation is about 16 ft/sec which is close to the theoretical celerity of $\sqrt{gD}$.

31. An important and useful method of portraying canal surge is by plotting the maximum negative surge profile along the canal as presented in Figure 13. The minimum elevation of this profile indicates the location of the maximum surge (except in the immediate vicinity of the lock) about 2900 ft upstream of the lock gates. This point is in the upstream end of the 1:10 transition, about 200 ft downstream from the smallest 150-ft-wide canal cross section. The maximum surge is about 1.45 ft. A tow on standby near that position would bump or strike the bottom as the surge passes.

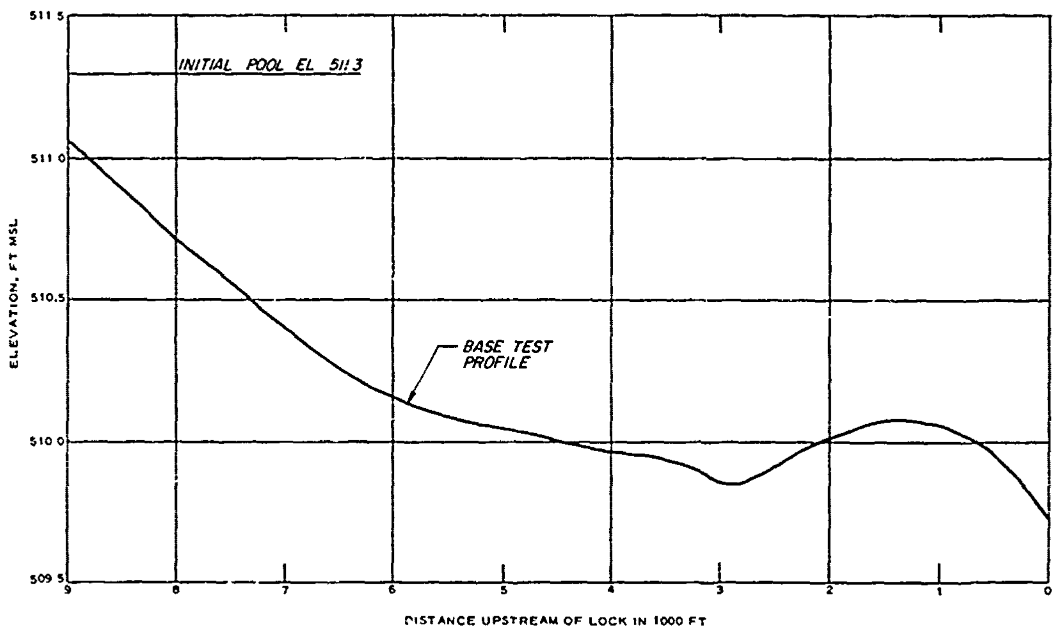

Figure 13. Maximum surge profile without tow

Effects of tow in canal

32. In some circumstances, lock filling operations may be necessary with a tow standing by in the canal. Computations were made with a 105-ft-wide by 600-ft-long tow centrally located at about 6700 ft upstream from the lock. The assumption was made that the downbound tow pilot would choose to stand by inside the narrow 150-ft-wide canal, but as far from the lock surge as possible, while waiting for the lock to fill. The tow was assumed to respond to the surge wave by following the free surface in a quasi-static manner. This is consistent with the long wave approximation which is the basis of the lock surge model. A small space step is necessary to resolve the tow effects on the numerical surge model. A total of nine space steps were used, that is, there was a different Δx in each of the nine branches (Table 2). The smallest Δx was in the branch simulating the tow.

33. The method of introduction of the tow effects on the negative surge was based on the "sunken tow" concept. The tow is assumed to act essentially as an abrupt contraction at the bow and an abrupt expansion

at the stern. Qualitatively, this is equivalent to decreasing the canal cross-sectional area by the tow cross-sectional area. Further elaboration of this concept is given in the paper by Kolkman (1978). For application to the canal surge computations, the tow cross-sectional area was subtracted from the canal area at the bottom of the canal.

34. Figures 14 and 15 present two time-histories of surge passage

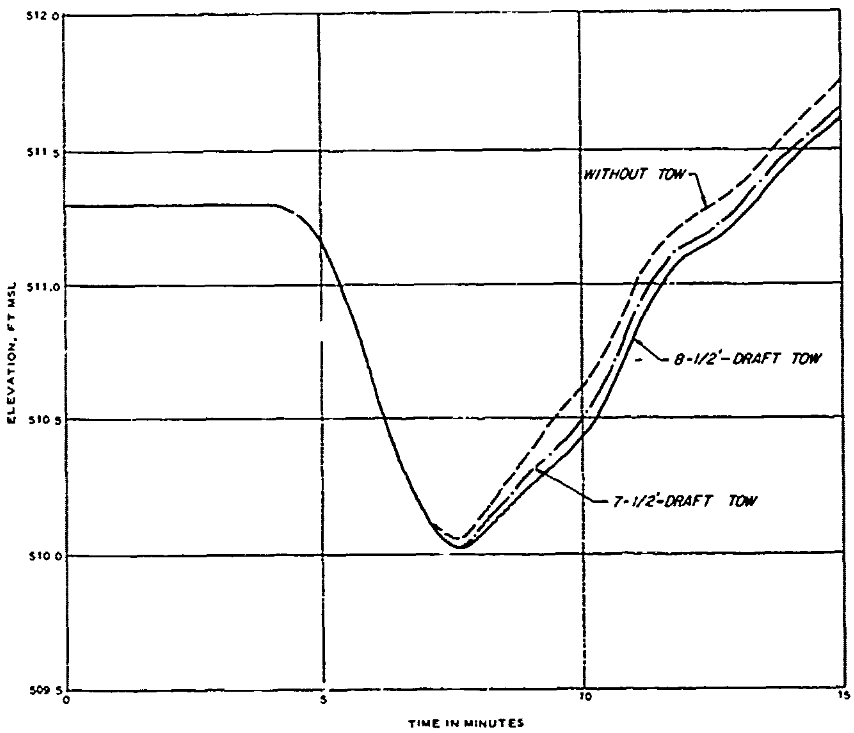

Figure 14. Effect of tow in canal at gage 4

downstream of the tow bow and at the midtow cross section. Two tow drafts of 7-1/2 and 8-1/2 ft were tried but apparently had only minor effect on the surge. The effect of an 8-1/2-ft-draft tow is to increase the negative lock surge at and between the tow and the lock. The maximum effect at the midtow location is about 0.2 ft.

35. The effect of the tow is more clearly shown in Figure 16 which portrays profiles of maximum negative surges. The maximum drawdown of the surge occurs at the bow of the tow, causing an increase in surge amplitude from 1.08 to 1.40 ft. The effect attributable to the tow is thus equal to about 30 percent of the maximum surge without the

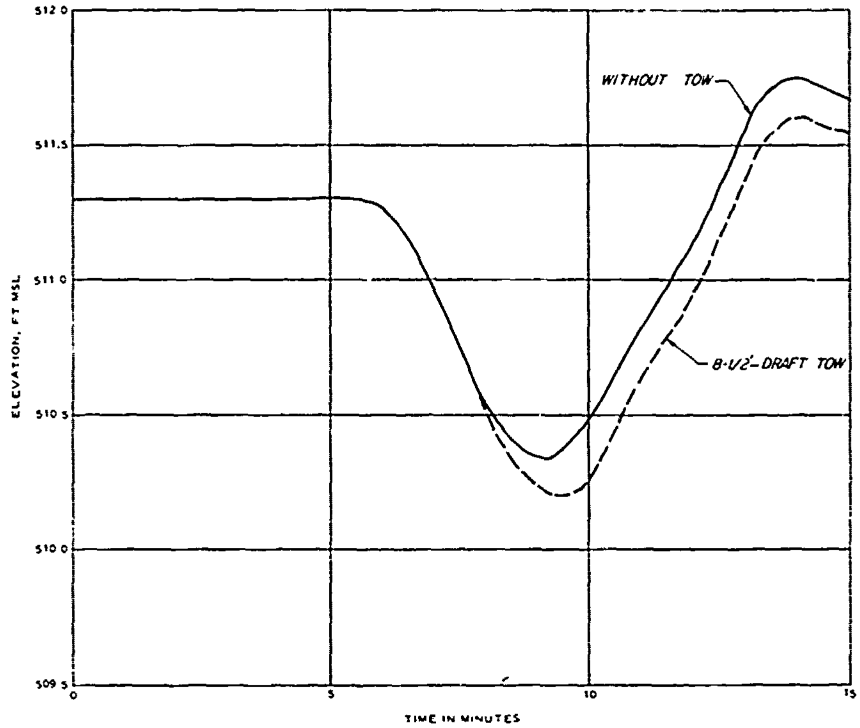

Figure 15. Effect of tow in canal at midtow

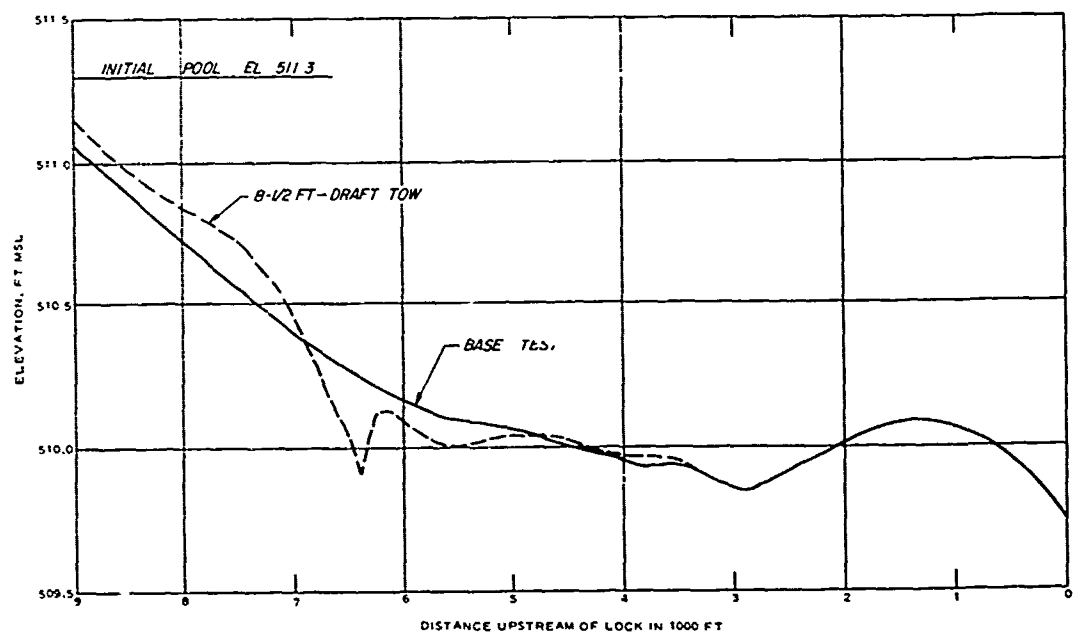

Figure 16. Maximum surge profile with and without tow

tow. This implies that a tow located at the lock surge minimum near the beginning of the 1:10 transition may be amplified to about 1.9 ft.

36. It had initially been planned to conduct additional computations to study the relative tow effect at several other canal sections. However, it was decided to concentrate more effort on tow squat effects, since this appeared to be more critical to the navigation problems.

Effects of Canal Modifications

37. A total of 15 canal configurations were tested with identical boundary and initial conditions. In all cases, the initial conditions used were el 511.3 (the observed elevation during the field tests) throughout the model and zero discharge. The 15 tests represented almost all the canal configuration combinations needed to obtain a complete picture of the impact of canal geometry changes (widening and/or deepening) on the lock surges. One combination for the 14-ft-deep by 300-ft-wide canal was inadvertently missed during generation of the canal geometry data tables.

38. Figure 5 shows the method of numerically "dredging" the canal at a typical cross section. Note that the existing canal is considerably larger than the trapezoidal designed canal of 9 ft deep, 150-ft bottom width, and 1-on-3 side slope. With a pool water-surface elevation of 511.3, the design canal at 9- and 12-ft depths (relative to normal pool el 511.0) would have the following numerical properties as compared with the actual canal measurements.

Depth ft	Depth at Test Pool el 511.3	A sq ft	W ft	$\overline{D}$ ft
9	9.3	1654.5	205.8	8.04
12	12.3	2298.9	223.8	10.27
Base*	Base	1984.2	238.4	8.32

* At the smallest section 8 (see Figure 4).

39. A summary of pertinent geometrical canal data for the test conditions is presented in Table 5. The canal area increase due to widening and/or deepening is also tabulated as an index of probable relative volume of canal dredging. For example, doubling the present canal bottom width to 300 ft will require a 69 percent increase in canal area.

40. The effect of canal widening alone is presented in Figure 17

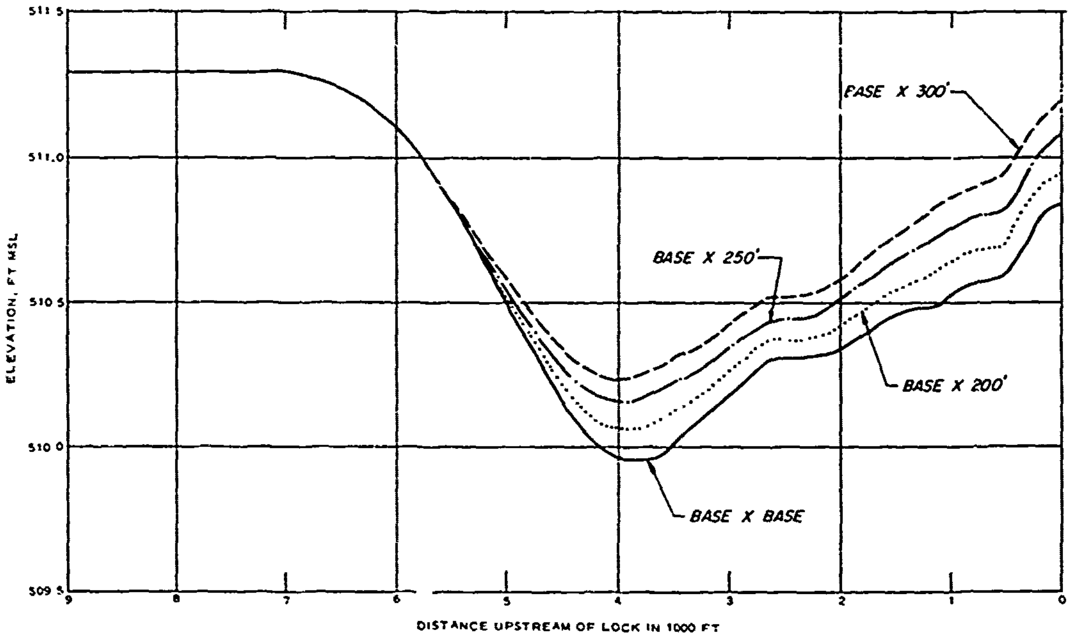

Figure 17. Surge profiles at end of lock filling (6 min), canal widened

showing surge profiles at the end of lock filling (6 min). Doubling the present nominal 150-ft-wide bottom width canal to 300 ft would decrease the surge amplitude by about 0.28 ft. The surge amplitude is decreased not only in the widened canal reach but also downstream in the reach from the 1:10 transition to the lock. The surge propagation speed is not changed materially by canal widening.

41. The effect of canal deepening alone indicates much more complex surge profiles as shown in Figure 18. The surge propagation speed is increased by depth increases but the maximum amplitude is only slightly decreased. The surge is decreased only by about 0.09 ft by

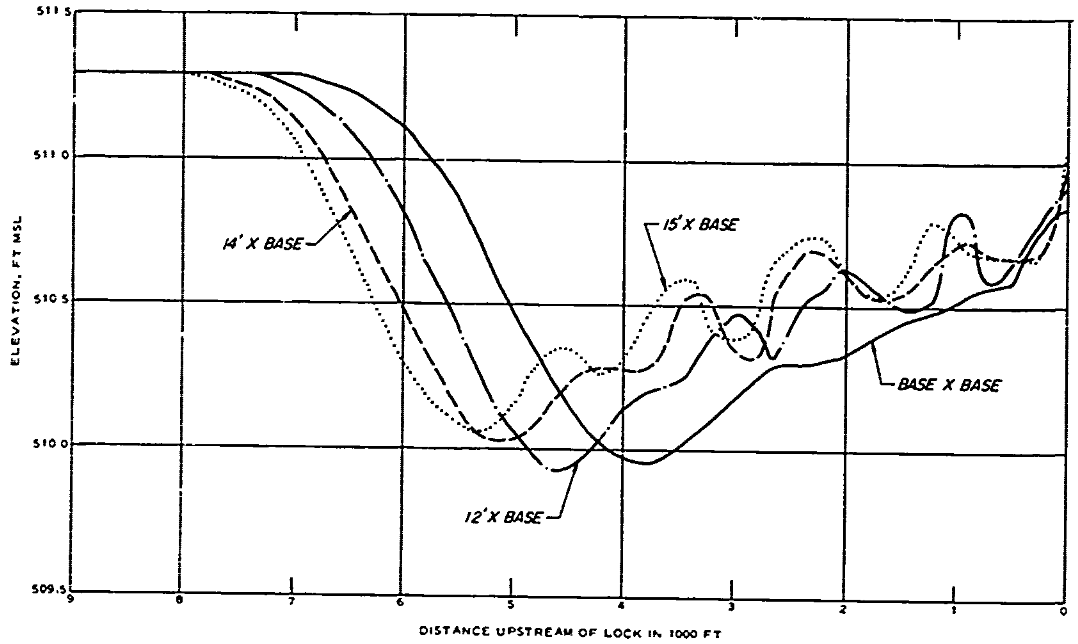

Figure 18. Surge profiles at end of lock filling (6 min), canal deepened

increasing the present canal to 15 ft. It appears, however, that deepening to 12 ft would actually increase the maximum surge by a very slight amount. It should be remembered, however, that the canal depth of 12 ft would result in an additional 3 ft of water under the tow. Thus, a negative surge of about the same magnitude as presently occurs would not result in grounding of a standby tow. Figure 19 presents the effects of doubling the present canal bottom width and deepening to 12 and 15 ft. Here again, the surge profile demonstrates the minor effect of deepening on surge amplitude. An increase in depth from 12 to 15 ft (25 percent increase) decreases the surge by about 0.1 ft.

42. Profiles of minimum canal elevations (or maximum surge amplitudes) for widened canals are presented in Figure 20. The largest surge amplitude in the profile (except in the immediate vicinity of the lock) occurs near the upstream end of the 1:10 transition (branch 8) for all three canals widened to less than 300 ft. These transition effects disappear at the widest canal width when the 300-ft width is constant along the entire length of the canal. Canal widening effects are

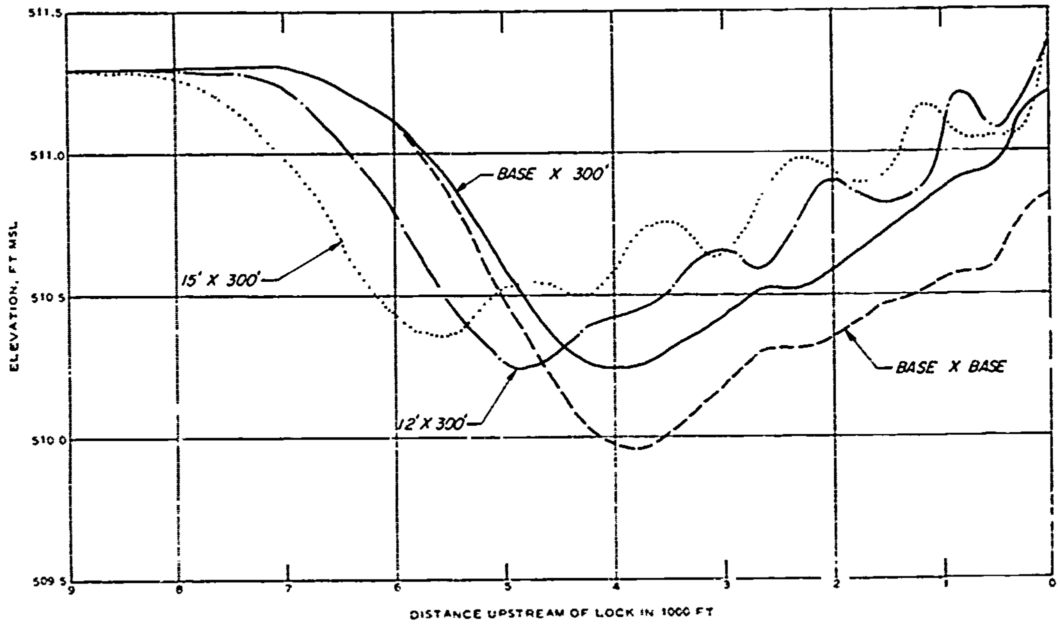

Figure 19. Surge profiles at end of lock filling (6 min), canal widened and deepened

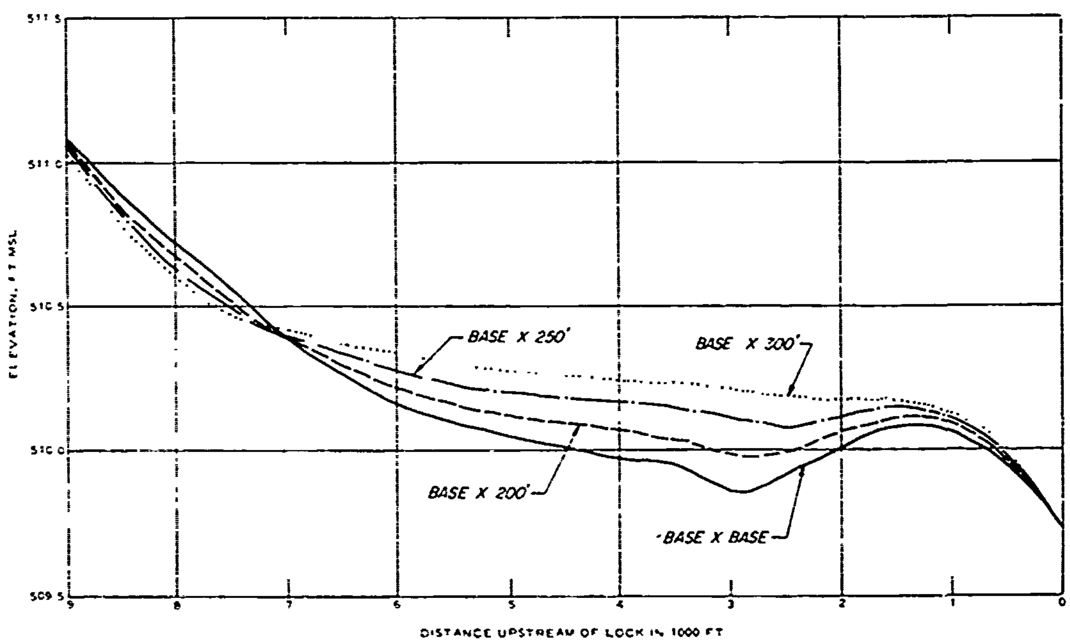

Figure 20. Maximum surge profiles; effects of canal widening

most pronounced in the reach where the canal is widened. The surge amplitude decreases as the width increases for most of the canal length.

43. The effects of increasing canal depth are shown in Figure 21.

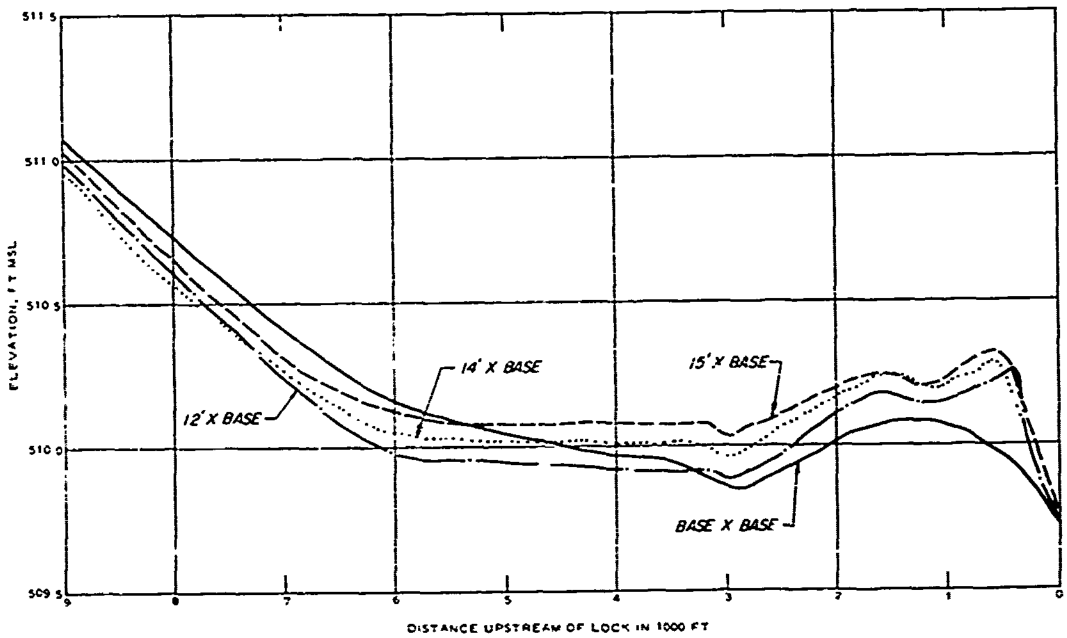

Figure 21. Maximum surge profiles; effects of canal deepening

Surge amplitudes are decreased by increasing depths in the downstream end of the canal from the 1:10 transition to the lock. Surge amplitudes are increased in the upstream half of the canal, but only by about 0.2 ft. Canal depth increases are less effective than width increases in surge reduction, but the additional depth would increase the clearance under a standby tow and prevent grounding.

44. Computational results with the canal widened to 300 ft and at various depths are shown in Figure 22. Surges are further reduced in the downstream end of the canal by deepening the 300-ft-wide canal. The largest canal cross section, 15 by 300 ft, provides the largest reduction in surge amplitude over most of the canal length. However, the surge amplitude for the largest canal excavated is still nearly 1 ft in magnitude (0.90 ft). This represents a decrease in surge amplitude of about 35 percent from the existing canal surge of 1.45 ft.

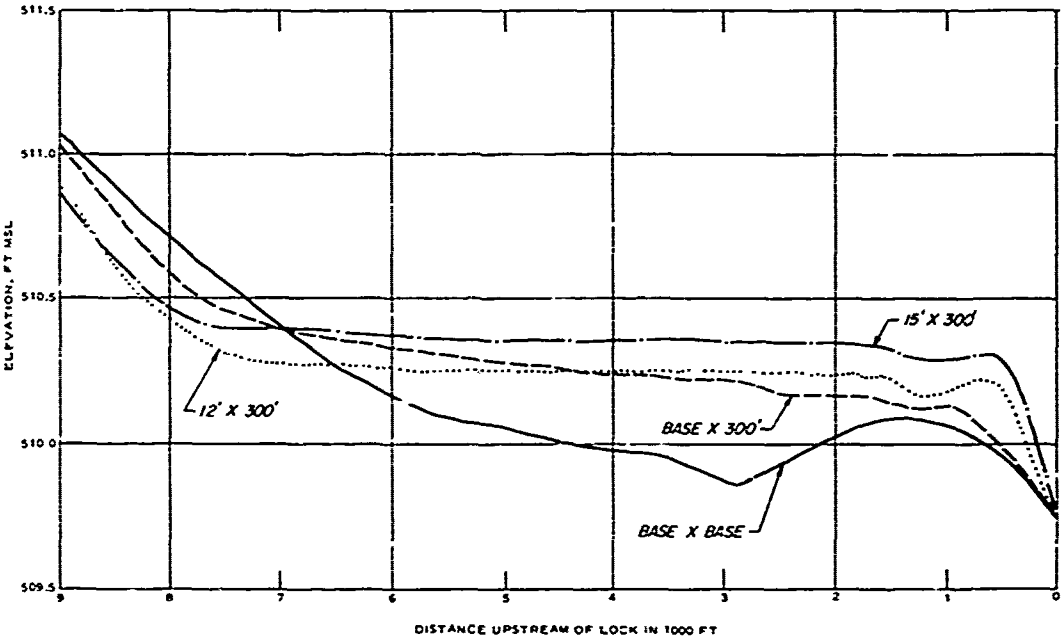

Figure 22. Maximum surge profiles; effect of deepening 300-ft-wide canal

Green's Law and Its Application

45. It is possible to study the lock filling surge in the approach canal using a relatively simple model of the phenomena. The intent is to find the relative impact of channel dimension modifications on lock surge and tow squat. The following paragraphs present the rationale and the theory of the method.

46. The bed slope of the approach canal, for all practical purposes, is zero. Resistance losses in the canal will be small during the propagation of the initial surge upstream from the lock chamber to the river. The detail derivation is presented in standard references on open-channel flow or wave motion (for example, Keulegan 1950).

47. As Figure 23 shows, the negative surge propagates upstream into the canal which has zero initial velocity. The amplitude of the negative surge is a function of the rate of lock filling and geometrical channel parameters. The Engineer Manual on lock surges (OCE 1949) suggests increasing the channel width and/or depth to reduce the surge

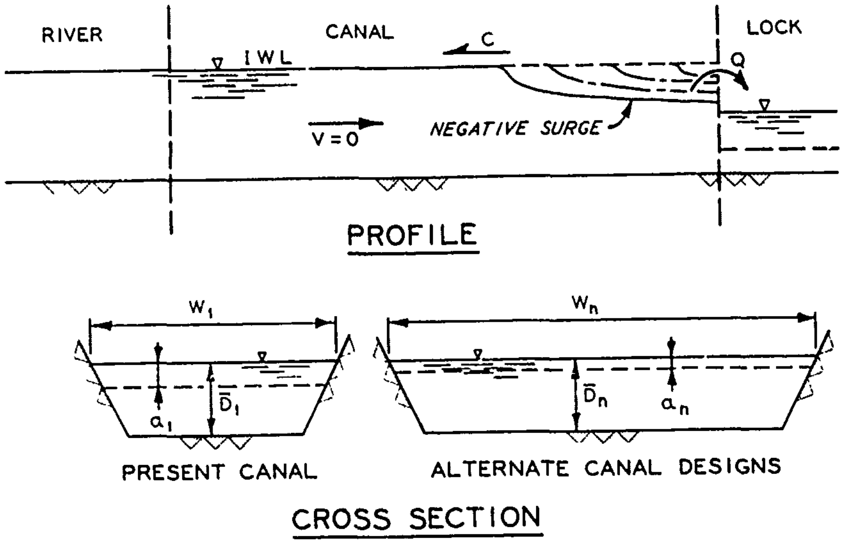

Figure 23. Lock surge in variable canal sections

height. Guidance on the relative benefits of channel modifications is not provided. It is possible to investigate the relative impact of canal geometry by means of the well-known Green's law. The law is a natural consequence of assuming equal surge energy for the alternative channel dimensions. The law states that the surge amplitude is proportional to the inverse of both the square root of canal width and the fourth root of the mean depth. In symbols, the relation is given as

$$\frac{a_n}{a_1} = \left(\frac{W_1}{W_n}\right)^{1/2} \left(\frac{\bar{D}_1}{\bar{D}_n}\right)^{1/4} \tag{14}$$

where

a_n = surge amplitude for any given variable canal geometry
a_1 = surge amplitude for the present unmodified canal
W_1 = present canal top width
W_n = variable geometry canal top width
$\bar{D}_1$ = A_1/W_1 present canal mean depth
$\bar{D}_n$ = variable geometry canal mean depth

48. The relative amplitudes can be normalized to obtain relative surge reductions by calculating $(a_1 - a_n)/a_1$ for ease of visualization of relative area increases. Table 5 presents values of relative surge amplitude reductions using Green's law. These are compared with the mathematical model results in Figure 24 showing reasonably accurate

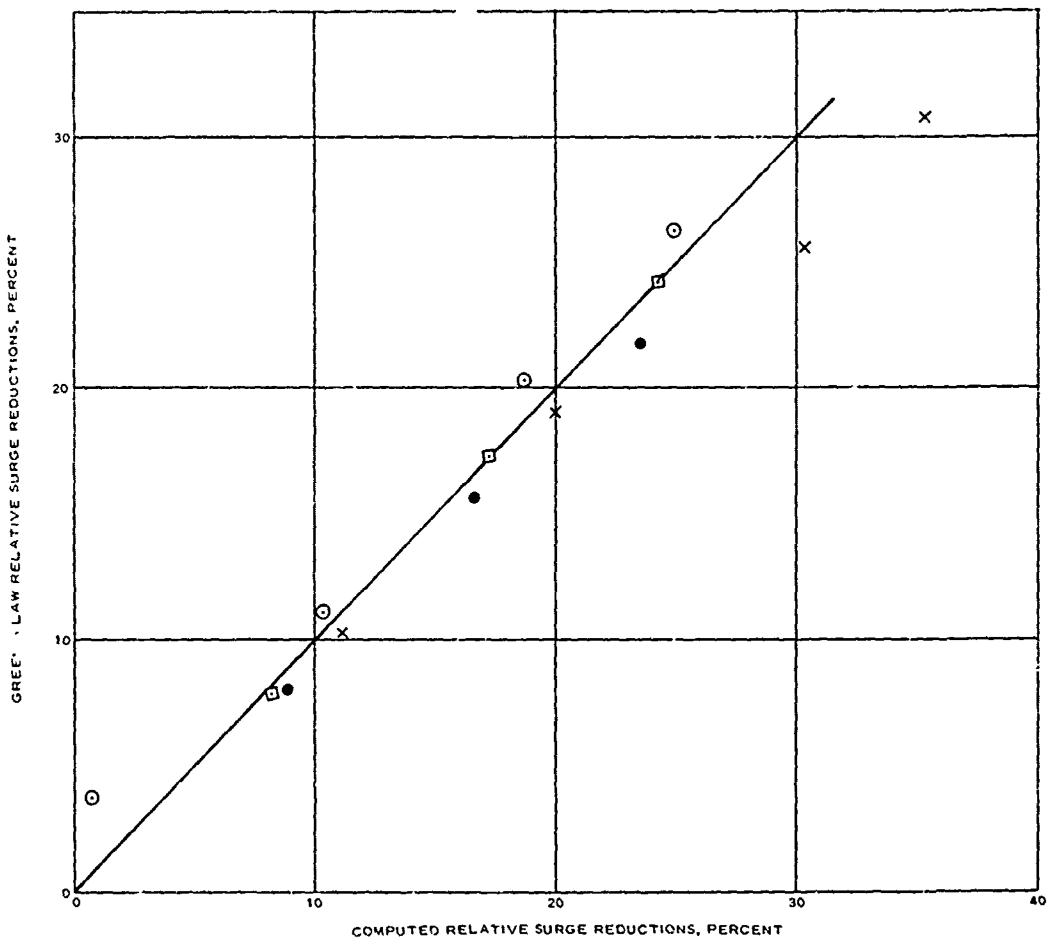

Figure 24. Comparison of model computations with Green's law

estimates of the impact of canal geometry changes on the surge by using Green's law. Figure 25 presents a generalized plot using Green's law to portray the surge amplitude reduction from canal modifications. The plot shows that canal widening reduces surge amplitude more than an equivalent area increase by canal deepening.

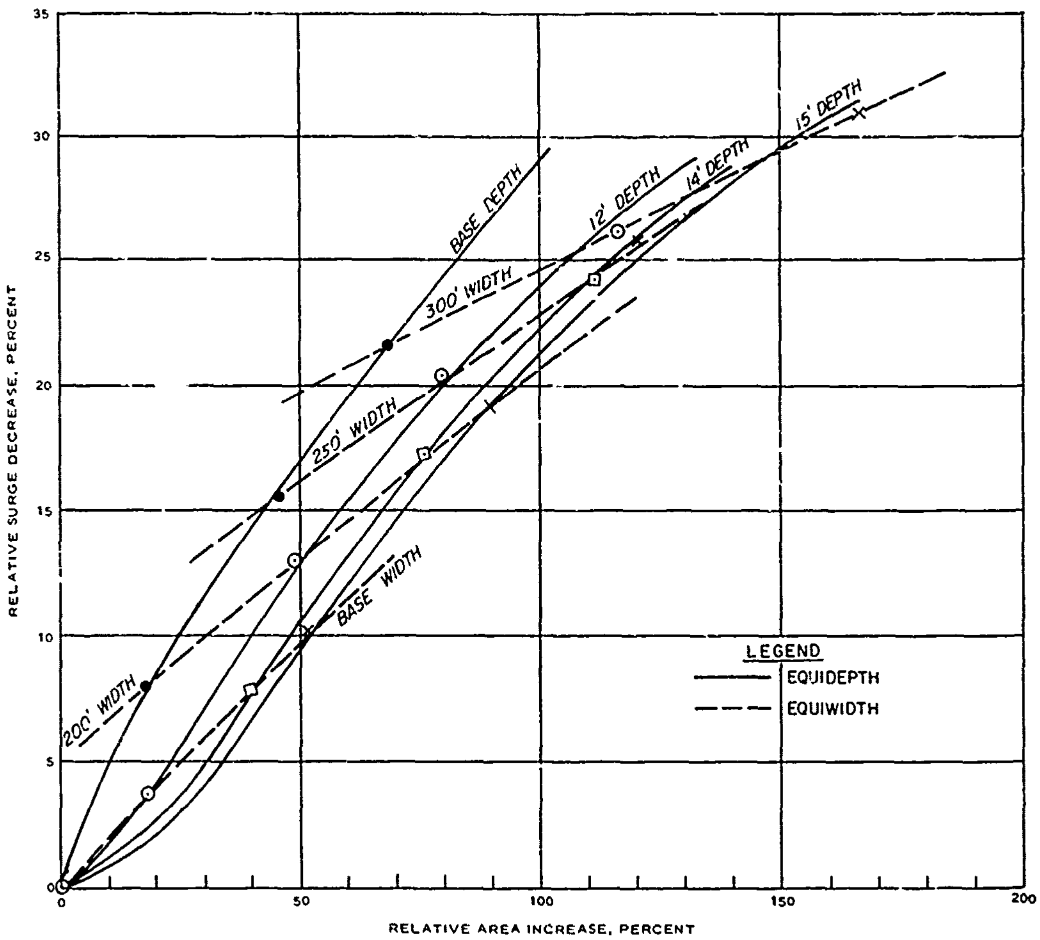

Figure 25. Relative lock surge reduction based on Green's law

49. Green's law assumes equal surge energy for canal geometry alternatives. This is a reasonable assumption in gradually varied canal geometries such as in the 1:10 transition. When canal area changes are not gradual, the surge analysis for abrupt expansions and contractions should be used as presented by Keulegan (1950) and in the Engineer Manual on canal surges (OCE 1949). This will result in higher surges than those determined by Green's law for equivalent canal area changes.

Summary of Lock Surge Results

50. Maximum negative surge amplitudes were abstracted from the

15 computational runs and are given in Table 6. Data are shown at the end of lock filling (6 min) and the maximum values from surge profiles are given. The maximum surge magnitudes near the upstream end of the 1:10 canal transition are shown plotted in Figure 26 as a function of

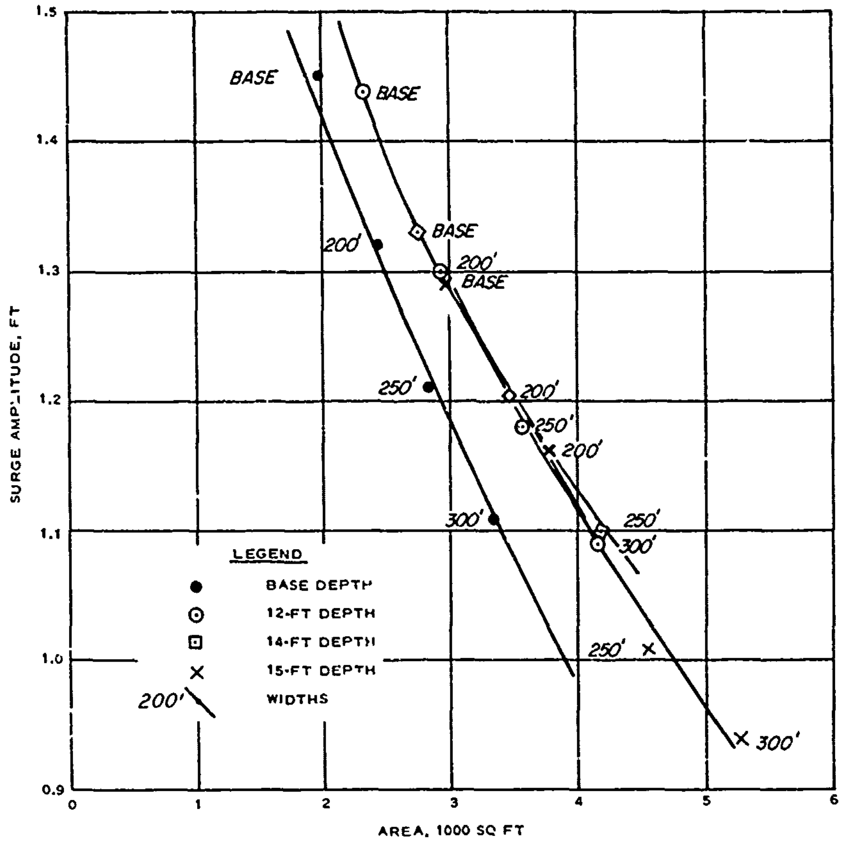

Figure 26. Summary of results of model computations

canal area. As expected, surge amplitude is decreased by increasing the canal area. Doubling the present canal cross-sectional area (bottom width of about 350 ft) without deepening will decrease the surge to about 1 ft. The rate of decrease of surge is greater due to canal width increases than by canal deepening. Increases of only canal width, however, do not increase the bottom clearance under the tow which is more critical at the time of maximum surge.

PART IV: TOW SQUAT APPLICATIONS

Steady Ahead

51. Computations were made using the Schijf squat model as described in PART II and are summarized in Table 7. Plotted squat results of these calculations will be discussed in subsequent paragraphs. All computations were made using the 8-1/2-ft draft by 105-ft-wide design tow.

52. Tow squat increases as the square of the tow speed as shown in Figure 27. This plot also presents the effect of widening the present canal and the Schijf limiting speed and the 90 percent limiting speed for each canal configuration. Squat is greatly increased when tow speeds become larger than 90 percent of the limiting speed. Laboratory and field tests have indicated that self-propelled tows cannot exceed V_ℓ and are usually operated from about 50 to 90 percent of V_ℓ. The squat in the present canal at $0.9V_\ell$ is about 0.7 ft which allows only about 0.1-ft clearance underneath the tow for a pool elevation of 511.3. Widening the canal increases the limiting tow speed. If it is assumed that tows will proceed at the highest possible speed of about 90 percent of V_ℓ, then increasing the canal width will not alleviate the tow grounding problems caused by squat.

53. The effects of increasing the canal depth are shown in Figure 28. Squat at limiting tow speed increases more rapidly with canal depth increases than with widening. The 12-ft canal depth (below 511 msl) will provide adequate depth to accommodate the 8.5-ft tow draft plus the 0.9-ft squat at $0.9V_\ell$.

54. The effect of canal width on the 12-ft-deep canal is presented in Figure 29. Tow squat is not a strong function of canal width at 90 percent of limiting speed. The maximum tow squat is about 1.0 ft for the 12- by 300-ft canal compared with 0.9-ft squat for the 12- by 150-ft canal. Either would give adequate clearance underneath the tow bottom.

55. Increasing canal area will allow the tows to proceed at a higher speed than is now possible in the existing canal which will be

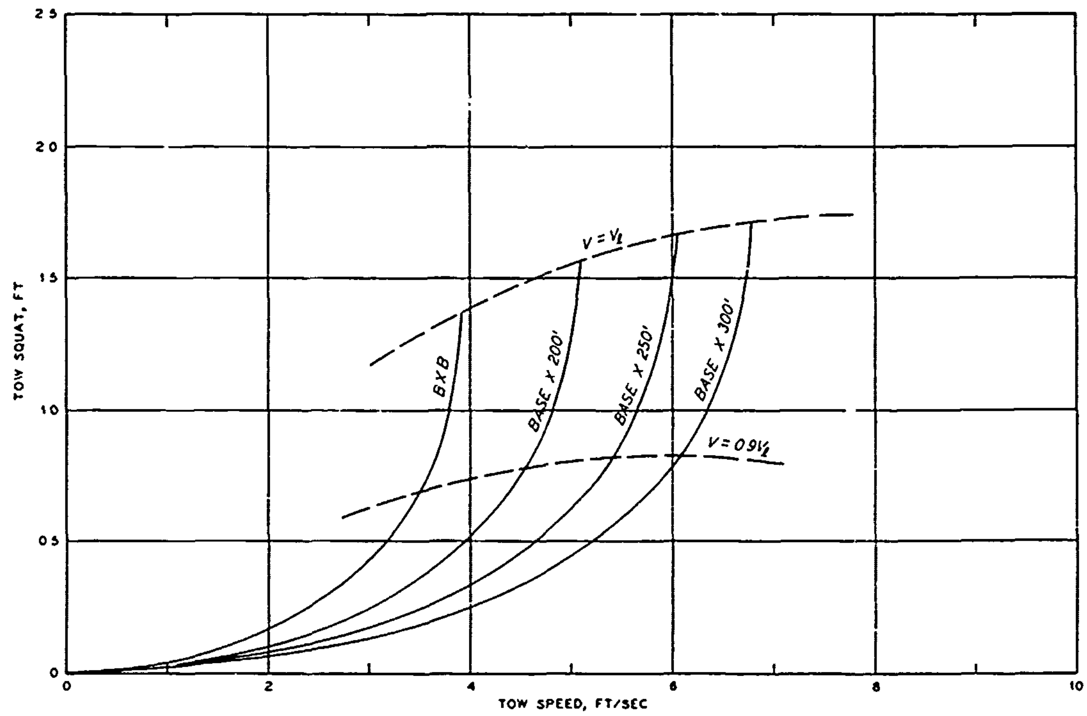

Figure 27. Effect of canal width increase on tow squat

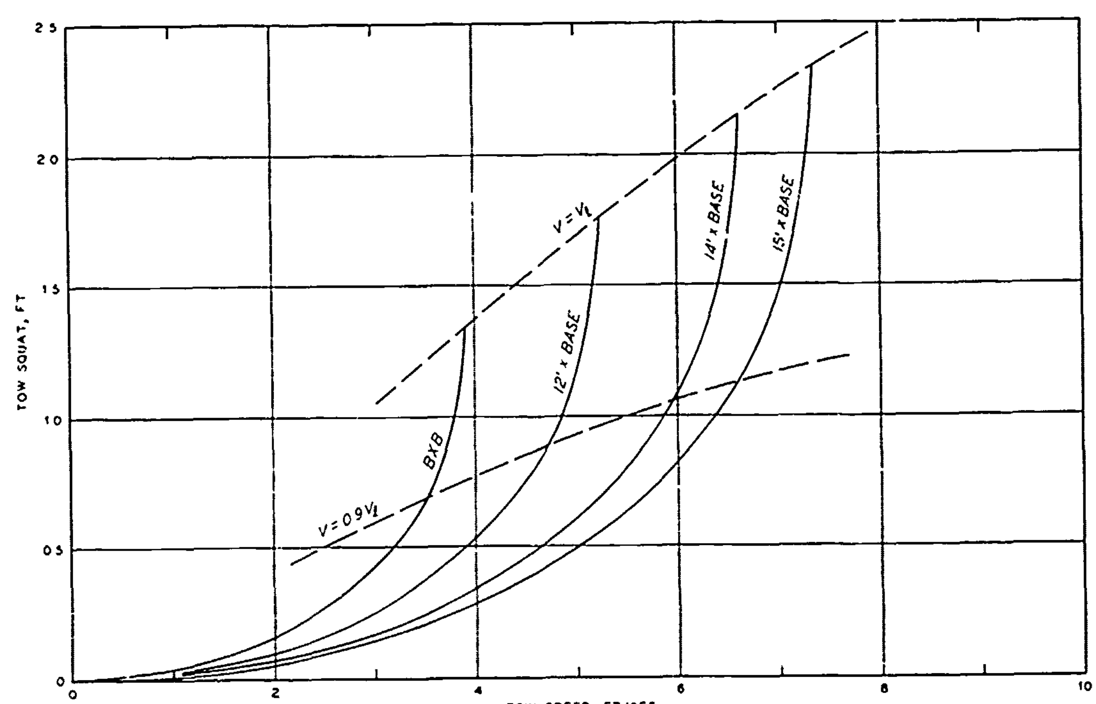

Figure 28. Effect of canal depth increase on tow squat

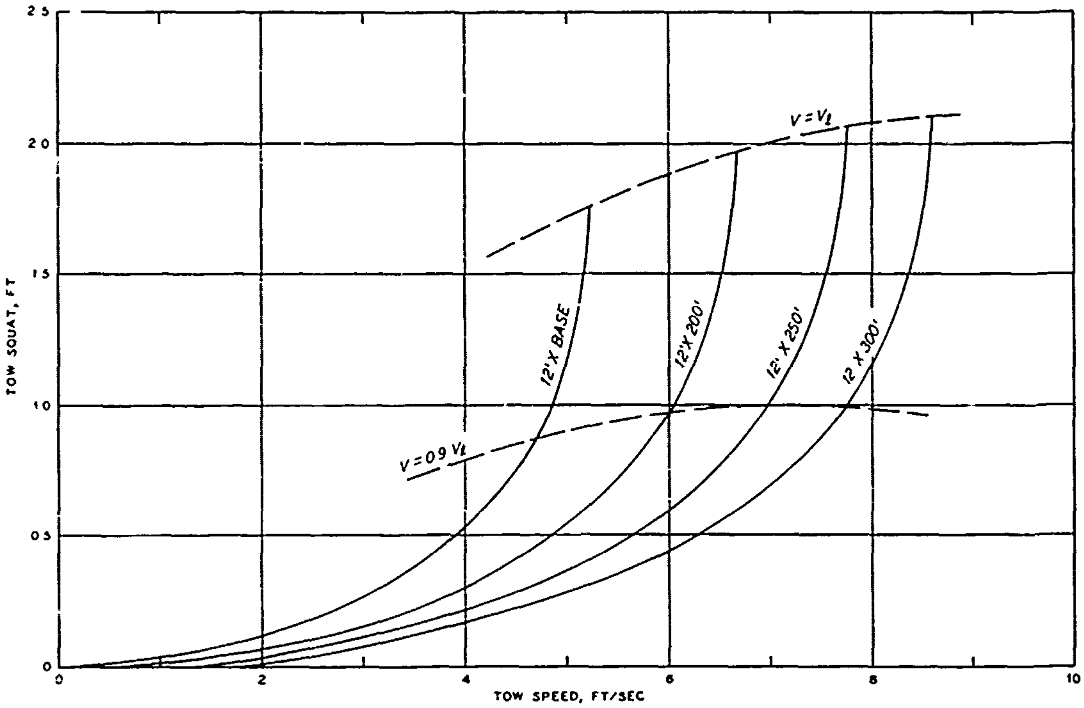

Figure 29. Effect of canal width increase on tow squat at 12-ft depth

of benefit to the towboat operations since this will reduce transit time through the constricted canal reach. Relative possible tow speed increase is shown plotted in Figure 30 as a function of relative canal area increase and is presented in Table 7. The graph shows that increasing canal depth has a greater impact on tow speed than increasing canal width.

56. Squat will increase if tows are operated at the same percentage of the limiting velocity. The results of computations to determine the relative squat at 90 percent V_ℓ are shown graphically in Figure 31 and are presented in Table 7. The plot shows that the relative squat increases more rapidly by deepening the canal than by widening it. Table 7 shows, however, that the increase in squat is small and less than the increase in canal depth; thus, it would be more advantageous to deepen the canal than to widen it, for a given increase in cross-sectional area.

57. Limiting the tow speed in the canal is one possible way of

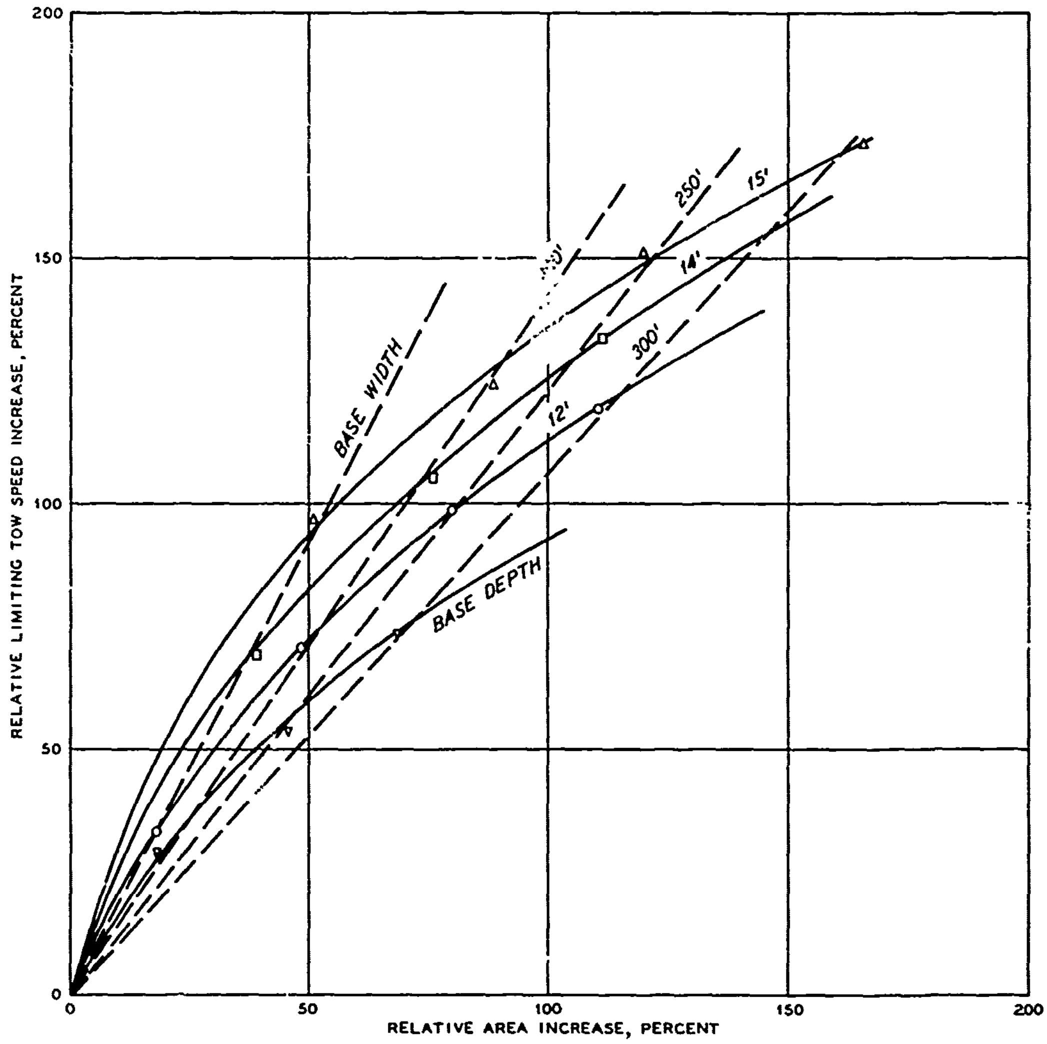

Figure 30. Effect of canal size on relative tow speed increase

greatly decreasing the risk of tow grounding. This is illustrated in Figure 32 which presents the relative decrease of squat due to canal area increases if the tow speed is held to the present maximum tow speed of $0.9V_{\ell}$ (this is approximately 2-1/2 mph). These data are also presented in Table 7. Canal deepening and widening have equal effect on squat decrease for a given increase in cross-sectional area.

Supersquat at Canal Transitions

58. Supersquat computations in the 1:2 transition were made using

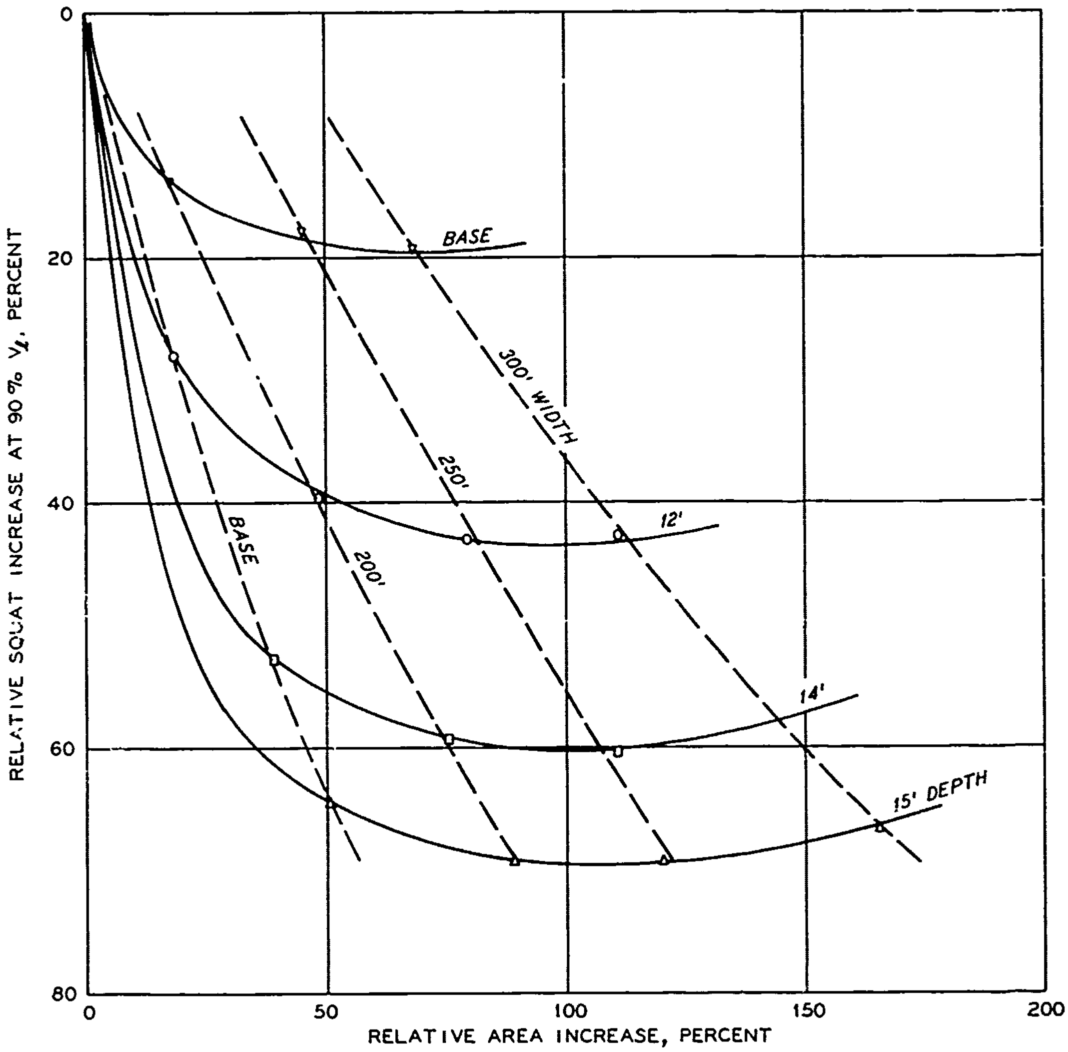

Figure 31. Effect of canal size on tow squat at 90 percent V_ℓ

the method presented in PART III for canal depths of 12, 14, and 15 ft at several canal widths in the constricted reach. As explained in the previous discussion of steady ahead squat, a canal depth of 12 ft would provide sufficient clearance to accommodate steady squat for any canal width at up to 90 percent of the limiting canal velocity. Results of the supersquat computations are presented in Tables 8, 9, and 10 and plotted in Figure 33. The relative supersquat effect in the canal transition indicates about an 18 percent increase in supersquat if the

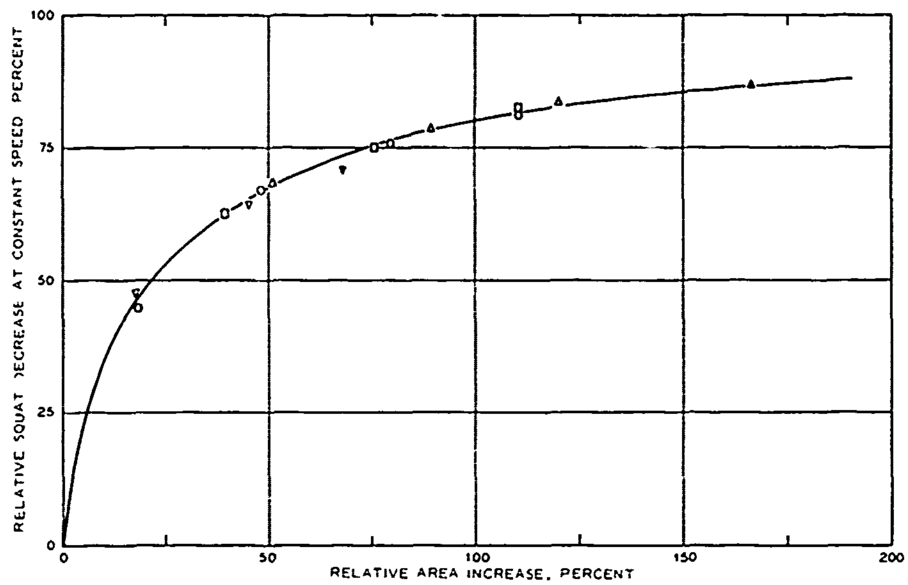

Figure 32. Effect of canal size on tow squat at 2-1/2 mph

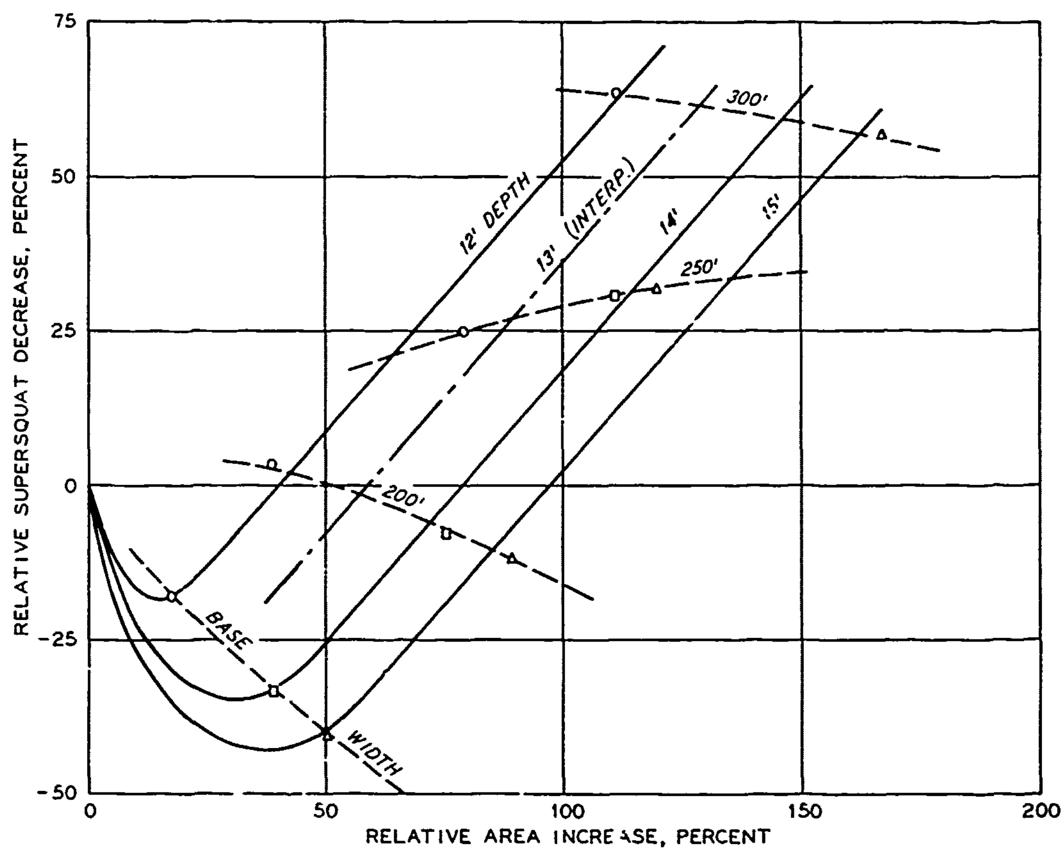

Figure 33. Effect of canal size on tow supersquat at 1:2 transition

present canal depth is increased to 12 ft without increasing the width; there would be about a 40 percent increase at 15-ft depth. Increasing the width to more than 200 ft is required to have a negligible supersquat change compared with the present canal. A canal width of greater than about 250 ft is necessary to reduce the present supersquat by more than one-fourth. The full width of 300 ft without a canal transition reduces the supersquat to essentially the same magnitude as the steady ahead squat of slightly less than 1 ft. Width increases are more effective in reducing relative supersquat effects compared with depth increases for equivalent canal area increases.

PART V: DISCUSSION OF RESULTS

59. The lock surge results indicate that surge magnitudes will remain appreciable in any of the canal configurations modeled compared with the existing canal. Surge magnitudes can only be reduced from about 1.5 ft to slightly less than 1 ft. A surge basin (or other possibly expensive remedial works) would be necessary to reduce the lock surge to a negligible amount. Increasing the lock filling time from the present 6 min would decrease the surge amplitude, but this might cause unacceptable delays as traffic increases in the future. The design and operation of canal and lock should accommodate the expected surge, since further surge reduction would probably be cost-prohibitive.

60. Steady squat in the canal depends on the square of the tow speed going through the canal. While the grounding problems could be eliminated by imposing speed limits, this is not usually found to be practicable. The analysis presented in this report was made by assuming that maximum possible tow speeds would be approached by some pilots. Minimum acceptable canal dimensions were defined as the dimensions which would accommodate the design tow under those conditions. Using these criteria, it was determined that the present canal is inadequate to handle the design tow. The steady squat computations indicate that canal depths of 12 ft or more would provide adequate bottom clearance. Canal deepening is more effective in increasing limiting tow speed than canal widening. A trial canal depth was thus set at 12 ft, with the possibility of increasing or decreasing the design depth above or below this value if warranted by further considerations.

61. Supersquat computations at the canal transition with several combinations of canal depths and widths showed that the canal width would have to be increased to at least 200 ft to not increase present supersquat effects. A uniform canal width of 300 ft would eliminate all supersquat effects at both canal transitions. The impact of the various alternative canal sizes on the net clearance between the canal and tow bottom is the main factor to consider for canal design. It is necessary, however, to consider the possibility of coincidental effects from lock

surges and supersquat from tows maneuvering through the canal transitions. The timing of lock surges and possible tow movements through these transitions suggests that surges and supersquat could coincide, especially in the upstream transition. An allowance of about 1.0 ft due to surge effects at and in the vicinity of the transitions seems reasonable based on a study of the surge profiles shown in Figures 20, 21, and 22. In addition, prudent design would suggest some minimum bottom clearance under the tow. If this amount is set at 0.5 ft, which is the bottom clearance for the present canal, then the last column in Table 11 can be used as a guide in alternate canal size selection. The table shows that a depth of 15 ft would be unnecessary and uneconomic. The following canal sizes give approximately equal clearances under the tow.

Depth, ft	Width, ft	Approximate Canal Area Increase, %
12	275	95*
13	200	62*
14	Base	39

* Interpolated from Table 5.

62. If only dredging costs are considered, the most economic solution may be to deepen the present canal to 14 ft. However, supersquat effects would be increased from the present canal conditions and the increase in positive surge would also increase the tow resistance in the transition. The increased surge conditions may cause an adverse reaction from navigation interests and is thus not recommended. The 13-ft-deep by 200-ft-wide canal size would not increase surge magnitudes above existing conditions. If costs can be justified by navigation traffic, the full 300-ft-wide by 12-ft-deep canal would significantly improve navigation conditions. The depth decrease from 13 to 12 ft could be achieved by altering maintenance dredging practice if the 13- × 200-ft section were adopted as an interim improvement. The recommended canal size increases would have the following impact on the main parameters involved in the choice of canal size.

Canal Parameter	Existing Unimproved Canal Base Depth by Base Width	Possible Intermediate Canal Improvement 13-ft Depth by 200-ft Width	Eventual Canal as Designed 12-ft Depth by 300-ft Width
Canal area, sq ft	2000	3200	4200
Canal to tow area ratio	2.2	3.6	4.7
Approx excav vol, M cu yd	0.00	0.20	0.35
Max surge amplitude, ft*	1.5	1.3	1.1
Maximum tow speed, mph	2.4	4.5	5.3
Steady squat at max speed, ft	0.7	1.0	1.0
Supersquat, ft	2.7	2.7	0.0
Net design clearance, ft**	Grounding	0.3	1.0

* Obtained from Table 6.

** Obtained from Table 11, assuming 1.0-ft surge amplitude at the upstream canal transition, the coincidence of tow supersquat, and a 0.5-ft minimum bottom clearance.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

63. The following conclusions and recommendations are based on the results of the numerical model computations presented in this report.

- a. An analysis of the surge computations shows that canal widening provides greater surge reduction than canal deepening for a given increase in cross-sectional area. The magnitudes of the reductions, however, are modest, being less than 30 percent decrease of the present surge with 100 to 150 percent canal area increase.
- b. Canal area increases do decrease surge magnitudes, but the largest area increase to more than twice the present area cannot bring the surge below about 1 ft.
- c. Steady squat analysis indicates that a 12-ft depth is the minimum necessary to eliminate grounding. Canal width increases without deepening would be detrimental and cause a greater tendency for tow grounding.
- d. The squat analysis shows more increase in limiting tow speed from canal deepening than widening.
- e. Steady squat values are judged to be acceptable (less than 1 ft) for a 12-ft canal depth.
- f. If feasible, speed control is one method to reduce squat. However, the penalty in time of transit is not attractive and the speed limit probably could not be enforced.
- g. Increasing the canal depth without width increase would increase the amount of supersquat at the transitions. However, the net underkeel clearance increase is greater than the supersquat increase.
- h. A 200-ft-wide by 12-ft-deep channel would provide about equal relative supersquat to the present canal but could still result in grounding under adverse surge and pool conditions. A 12- by 300-ft canal reduced the magnitude of the present supersquat to steady ahead squat values and would eliminate the possibility of grounding.
- i. Optimum canal to tow clearance can be achieved with either of the following minimum canal configurations:

Depth, ft	Width, ft
12	275
13	200
14	Base

- j. A 200-ft-wide canal with a minimum 13-ft depth below el 511.0 to reduce grounding problems to an acceptable level

would provide adequate safety with minimum dredging cost.

k. It is recommended that canal size be based not only on excavation cost but also that consideration be given by the Tulsa District to maintenance dredging cost, mobilization cost for future required widening, and benefits from greater tow speed.

l. If navigation traffic can justify the additional cost, a 12-ft-deep by 300-ft-wide canal should be dredged. This would significantly improve limiting tow speeds, probably reduce transit times through the canal, and further improve navigation conditions.

m. It is important that the present canal not be widened without deepening as this would aggravate the grounding problem.

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Table 1

Main Cross-Sectional Elements

(at Pool El 511.0)

Cross Section*	Conveyance Top Width ft	Left Bank to ℄ Nav Chan ft	Max Depth ft	Storage Top Width ft	Min Clearance Under 8-1/2-ft Tow** ft
1	569	178	34.6		14.5
2	522	177	28.9		16.5
3	511	174	24.2		14.5
4	390	121	12.0		1.8
5	370	118	12.3		0.5
6	236	122	12.8		2.0
7	232	120	12.0		0.7
8	236	120	11.9		0.6
9	380	125	11.3		0.8
10	325†	55†	24.1		12.5
10-1	243	55	24.1		
10-2	177	55	24.1		
10-3	110	55	25.0		
10-4	110	55	25.0		
RB-1	414	--	27.8		
RB-2	352	--	30.7		
LB-1			3.8	587	
LB-2			3.4	114	
LB-3			4.4	186	
LB-4			6.3	422	
Boat ramp			9.8	355	

* See Figure 4 for location of cross sections.

** Tow static draft of 8-1/2 ft; 105-ft beam tow on ℄ navigation channel.

† Section to face left lock wall. Total section width approximately 468 ft.

Table 2

Summary Table of Schematization

Branch No.	Total Length ft	Grid (Δx) ft	No. of Grids	Name	Notes
1	2,985	186.5625	16	River	Upstream boundary condition
2	1,257	157.125	8	Spillway	Upstream boundary condition
3	1,416	177.00	8	Canal entrance	Tee junction
4	900	75.00	12	1:2 transition	
5	600	60.00	10	Tow	
6	2,974	185.875	16	Canal	
7	1,500	150.00	10	1:10 transition	
8	1,080	180.00	6	Wide canal	
9	521	130.25	4	Lock intake	Downstream boundary condition
TOTAL	13,233		90		

Table 3

Location of Model Elements

Station ft	Cross Section	Gage	Storage	Junction	Transition	Notes
				River/Canal/Lock		
2227 + 90						End curve downstream bridge
2229 + 10	1			1		Upstream model limit
2238 + 55		7				Downstream old Hwy 69 Bridge
2239 + 35	2					
2244 + 64			1			Beg 200' lt bank
2246 + 64			1			End storage
2248 + 64	3					
2258 + 80			2			Beg 450' lt bank
2258 + 95				2	24' depth to 12'	Spillway branch No. 2 Rt bank
2261 + 83						Beg canal - 300' bottom width
2263 + 30			2			End storage
2265 + 20	4					
2269 + 41		5				300' bottom width canal
2270 + 11=						Start 13^o curve rt
2246 + 08						
2248 + 98	5					
2249 + 08				4	1:2	Beg 300' to 150' bottom width
2252 + 08					1:2	End 300' transition
2252 + 34	6					
2258 + 08				5	Canal/Tow	Stern tow position
2264 + 08				6	Tow/Canal	Bow 600' tow
2268 + 83	7					
2269 + 88						End 13^o curve
2273 + 33		4				150' bottom width canal
2292 + 90	8					
2293 + 82				7	1:10	Beg 150' to 300' bottom width
2298 + 10		3				In transition
2308 + 82				8	1:10	End 1500' transition
2309 + 05	9					
2319 + 62				9	Canal/Lock Intake	End 300' bottom width canal Beg 521' lt and rt eddy zones
2320 + 82						Beg lt guide wall
2320 + 92	10-1		4,5			Lt and rt eddy zones
2372 + 00	10					
2322 + 23	10-2					
2322 + 71						Beg rt guide wall

(Continued)

Table 3 (Concluded)

Station ft	Cross Section	Gage	Storage	Junction	Transition	Notes
			River/Canal/Lock (Continued)			
2323 + 53	10-3				110' × 25'	Lock chamber intakes
			5			End 521' eddy zones
2323 + 80		2				Rt bank in eddy zone
2324 + 83	10-4			10		Downstream model limit
2325 + 23						℄ upstream pintle
2325 + 74		1				Lock chamber
			River/Spillway			
Distance ft						
0				2		Beg branch No. 2
170			3			Beg 560' rt bank
260	RB-1					
730			3			End boat slip storage
840	RB-2					
1068		6				Rt bank spillway
1240						Axis of dam
1257				3	180' × 25'	Spillway gates-end branch No. 2
1295						Gate trunnion

Table 4

Ship or Tow Squat Parameters in Canals by Schijf Method

(Using HP65)

Key Entry	Comments	Key Entry	Comments	Registers	Label
LBL	Computes V_ℓ, u_ℓ, & z_ℓ	LBL	Computes u and z for	R_1: $n = A/a$	
A	at limiting velocity	B	velocity V_t less than		
1		1	limiting velocity		
RCL1	= n	RCL1	= n	R_2: $\bar{D} = A/W$	
g		RCL5	$= z_k$		
1/x		X			
−		RCL2	$= \bar{D}$	R_3: LBLA $\sqrt{g\bar{D}}$, LBLB $V_t^2/2g$	
g		÷			
RAD		+			
f^{-1}		↑		R_4: z_k	
cos		↑			
g		RCL1	= n		
π		−		R_5: Temp Sto; LBLA $\frac{}{\text{Cos}}[\ ]$, LBLB z_{k+1}	
+		÷			
3		CHS	$= -\left[\dfrac{1 + \dfrac{nz_k}{\bar{D}}}{1 + \dfrac{nz_k}{\bar{D}} - n}\right]$		
÷		STO7			
f		2		R_6: V_ℓ	
cos		g			
STO5	$= \cos\left[\frac{\pi}{3} + \frac{\cos^{-1}}{3}\left(1 - \frac{1}{n}\right)\right]$	y^x		R_7: LBLA z_ℓ, LBLB $-[\ ]$	
↑		RCL7			
↑		2			
3		X		R_8: Temp Sto; LBLA $4\cos^3[\ ]$, LBLB V	
g		+			
y^x		RCL3	$= V_t^2/2g$		
4		X			
X		STO4	$z_{k+1} = \frac{V_t^2}{2g}\left\{[\ ]^2 + 2[\ ]\right\}$		
STO8	Temp Sto	1			
−		RCL5			
RCL2	$= \bar{D}$	RCL4			A | $V_t = V_\ell$
X		÷			B | $V_t < V_\ell$
STO7	$= z_\ell$ squat in ft	−			
RCL8		g			
2		ABS			
X		↑			
f		EEX			
√	$\sqrt{8\cos^3[\ ]}$	4			2 Branch to u
STO		CHS			
RCL		$gx \leq y$			
2		$gx < y$			
X		GTO			
f		2			
√	$\sqrt{2\cos[\ ]}$	RCL4			
RCL6		STO5	$= z_{k+1}$, squat in ft		
−		B			
RCL3	$= \sqrt{g\bar{D}}$	LBL			
STO		2			
X		RCL7			
6	V_ℓ, Limiting velocity in ft/sec	RCL8	$= V_t$		
X		X	$u_{k+1} = V_t[\ ]$		
RTN	u_ℓ, Return velocity in ft/sec	RTN	= Return velocity in ft/sec		

Table 5

Summary of Pertinent Canal Cross-Sectional Data

(Geometry Given at Minimum Area Section

No. 8 for Pool el 511.3)

Nominal Dimensions*							
Depth** ft	Bottom Width ft	Area A sq ft	Top Width W ft	$\bar{D} = A/W$ ft	$\sqrt{gD}$ ft/sec	Relative Area Increase† %	Green's Law $1 - (a_n/a_1)$ %
Base	Base	1984.2	238.4	8.32	16.37	0.00	0.00
Base	200	2412.7	273.4	8.82	16.86	17.76	7.98
Base	250	2880.7	323.4	8.91	16.94	45.18	15.59
Base	300	3347.7	373.4	8.97	16.99	68.72	21.57
12	Base	2342.0	235.1	9.96	17.91	18.03	3.73
12	200	2948.5	280.1	10.53	18.41	48.60	13.01
12	250	3563.5	330.1	10.80	18.64	79.59	20.37
12	300	4178.5	380.1	10.99	18.81	110.59	26.13
14	Base	2760.2	238.4	11.58	19.31	39.11	7.92
14	200	3476.2	289.4	12.01	19.67	75.19	17.19
14	250	4191.2	339.4	12.35	19.94	111.23	24.06
15	Base	2999.4	242.4	12.37	19.96	51.16	10.19
15	200	3764.4	292.4	12.87	20.36	89.72	19.04
15	250	4529.4	342.4	13.23	20.64	120.27	25.69
15	300	5294.4	392.4	13.49	20.84	166.83	30.92

* Base (existing) depth is 9 ft and base width is 150 ft.

** Nominal depth at normal upper pool el 511.0.

† Computed for each canal configuration compared with existing canal area of 1984.2 sq ft.

Table 6

Summary of Surge Calculations at Various Canal Conditions

Canal Test Cond	Min El @ 6 Min ft msl	Surge Amp ft	Surge Amp Reduction ft	Rel Amp Reduction %	Min El @ Sta 77/78 ft msl	Surge Amp ft	Surge Amp Reduction ft	Rel Amp Reduction %
Base × Base	509.96	1.34	0.00	0.00	509.85	1.45	0.00	0.00
Base × 200	510.06	1.24	0.10	7.46	509.98	1.32	0.13	8.97
Base × 250	510.17	1.13	0.21	15.67	510.09	1.21	0.24	15.55
Base × 300	510.25	1.05	0.29	21.64	510.19	1.11	0.34	23.45
12 × Base	509.93	1.37	-0.03	-2.24	509.86	1.44	0.01	0.7
12 × 200	510.06	1.24	0.10	7.46	510.00	1.30	0.15	10.34
12 × 250	510.18	1.12	0.22	16.42	510.12	1.18	0.27	18.62
12 × 300	510.25	1.05	0.29	21.64	510.21	1.09	0.36	24.83
14 × Base	510.04	1.26	0.08	5.97	509.97	1.33	0.12	8.28
14 × 200	510.14	1.16	0.18	13.43	510.10	1.20	0.25	17.24
14 × 250	510.23	1.07	0.27	20.15	510.20	1.10	0.35	21.14
15 × Base	510.07	1.23	0.11	8.21	510.01	1.29	0.16	11.03
15 × 200	510.19	1.11	0.23	17.16	510.14	1.16	0.29	20.00
15 × 250	510.28	1.02	0.32	23.88	510.29	1.01	0.44	30.34
15 × 300	510.35	0.95	0.39	29.10	510.36	0.94	0.51	35.17

Table 7

Summary of Squat Calculations at Various Canal Conditions

(Geometry at Section 8 - See Table 5)

Canal Test Conditions	Limiting Tow Speed V_ℓ ft/sec	Relative Limiting Tow Speed Increase %	Return Velocity u_ℓ ft/sec	Squat at V_ℓ ft	90% X V_ℓ ft/sec	Squat at $0.9V_\ell$ ft	Relative Increase in Squat at $0.9V_\ell$ %	Constant Speed* Squat ft	Relative Constant Speed Squat Decrease %
Base × Base	3.925	0.00	6.245	1.367	3.533	0.697	00.00	0.697	00.00
Base × 200	5.098	29.87	6.217	1.584	4.588	0.793	13.82	0.366	47.55
Base × 250	6.031	53.63	5.973	1.673	5.427	0.822	17.99	0.248	64.41
Base × 300	6.780	72.72	5.729	1.716	6.102	0.829	19.03	0.188	73.01
12 × Base	5.249	33.73	6.647	1.770	4.725	0.889	27.52	0.381	45.33
12 × 200	6.681	70.20	6.451	1.985	6.013	0.973	39.63	0.231	66.83
12 × 250	7.759	99.67	6.160	2.074	6.983	0.995	42.81	0.166	76.22
12 × 300	8.618	119.54	5.885	2.113	7.756	0.994	42.70	0.129	81.47
14 × Base	6.311	68.93	6.890	2.156	5.968	1.065	52.78	0.259	62.85
14 × 200	8.052	105.13	6.551	2.305	7.247	1.109	59.18	0.171	75.48
14 × 250	9.149	133.07	6.231	2.373	8.234	1.116	60.22	0.127	81.71
15 × Base	7.343	87.06	6.959	2.339	6.609	1.145	64.26	0.220	68.49
15 × 200	8.773	123.50	6.605	2.477	7.896	1.171	60.47	0.150	78.54
15 × 250	9.870	151.44	6.270	2.532	5.883	1.179	69.27	0.113	83.74
15 × 300	10.743	173.67	5.969	2.545	9.668	1.161	66.62	0.0912	86.92

* Constant speed used is equal to $0.9V_\ell$ for the existing (9' × 150') canal which is 2.41 mph.

Table 8

Summary of Supersquat Calculations at 1:2 Transition for 12-ft Depth and Various Widths

Variable	Base × Base		12' × Base		12' × 200'		12' × 250'		12' × 300'	
	Section Numbers									
	31	41	31	41	31	41	31	41	31	41
Area, sq ft = A	3694.7	2196.9	4248.2	2372.2	4243.2	2985.3	4248.2	3603.3	4248.2	4220.3
Width, ft = W	387.2	238.4	390.5	238.7	390.5	286.1	390.5	336.1	390.5	386.1
n = A/a*	4.140	2.462	4.760	2.658	4.760	3.345	4.760	4.037	4.760	4.729
$\bar{D}$ = A/W	9.542	9.215	10.879	9.938	10.879	10.434	10.875	10.721	10.879	10.931
$\sqrt{g\bar{D}}$, ft/sec	17.528	17.226	18.716	17.888	18.716	18.330	18.716	18.580	18.715	18.760
u_t, ft/sec	5.722	6.473	5.820	6.622	5.820	6.400	5.820	6.117	5.820	5.848
z_t, ft	1.835	1.595	2.090	1.774	2.090	1.971	2.090	2.061	2.090	2.100
V_t, ft/sec	7.460	4.699	8.651	5.315	8.651	6.718	8.651	7.788	8.651	8.641
$0.9V_t$, ft/sec	6.719	4.229	7.786	4.784	7.786	6.046	7.786	7.010	7.786	7.776
$z_{0.9V_t}$, ft	0.877	0.806	0.981	0.889	0.981	0.965	0.981	0.987	0.981	0.987
$u_{0.9V_t}$, ft/sec	3.362	4.124	3.342	4.170	3.342	3.889	3.342	3.608	3.342	3.361
$0.9V_{t(31)} - V_{t(41)}$	2.020		2.471		1.068		-0.002		-0.855	
η**	1.081		1.373		0.608		0.0		0.0	
$\Delta h = \eta + z_{t(41)}$ †	2.676		3.147		2.579		2.061		0.957	
$1 - \frac{\Delta h_n}{\Delta h_1}$, %	0.00		-17.60		3.63		22.98		63.12	

* a = Design tow cross-sectional area = 8.5 ft × 105 ft = 892.5 sq ft.

** $\eta = \bar{D}_{(41)} / \sqrt{g\bar{D}}_{(41)} \left(0.9V_{t(31)} - V_{t(41)}\right)$.

† If $\eta < 0$, $\Delta h = z_{(41)}\left(\text{for } V_t - 0.9V_{t(31)}\right)$.

Table 9

Summary of Supersquat Calculations at 1:2 Transition for 15-ft Depths and Various Widths

Variable	15' × Base		15' × 200'		15' × 250'		15' × 300'	
	Section Numbers							
	31	41	31	41	31	41	31	41
A	5330.2	3002.0	5330.2	3767.0	5330.2	4532.0	5330.2	5297.0
W	400.5	245.1	400.5	295.1	400.5	345.1	400.5	395.1
n = A/a*	5.972	3.364	5.972	4.221	5.972	5.078	5.972	5.935
$\overline{D}$ = A/W	13.309	12.248	13.309	12.765	13.309	13.132	13.309	13.407
$\sqrt{g\overline{D}}$	[illegible]	19.859	20.701	20.274	20.701	20.563	20.701	20.777
u_ℓ	5.913	6.922	5.913	6.575	5.913	6.245	5.913	5.949
z_ℓ	2.508	[illegible]	2.508	2.456	2.508	2.514	2.508	2.528
V_ℓ	10.701	7.310	10.701	8.740	10.701	9.837	10.701	10.711
$0.9V_\ell$	9.631	6.579	9.631	7.866	9.631	8.853	9.631	9.640
$z_{0.9V_\ell}$	1.143	1.133	1.143	1.171	1.143	1.171	1.143	1.154
$u_{0.9V_\ell}$	3.268	4.203	3.268	3.851	3.268	3.548	3.268	3.291
$0.9V_{\ell(31)} - V_{\ell(41)}$	2.231		0.891		-0.206		-1.08	
η**	1.431		0.561		0.0		0.00	
$\Delta h = \eta + z_{\ell(41)}$ †	3.752		3.017		1.827		1.154	
$1 - \frac{\Delta h_n}{\Delta h_1}$	-40.23		-12.74		31.74		56.88	

* a = design tow cross-sectional area = 8.5 ft × 105 ft = 892.5 sq ft.

** $\eta = \overline{D}_{(41)} / \sqrt{g\overline{D}}_{(41)} \left(0.9V_{\ell(31)} - V_{\ell(41)}\right)$.

† If $\eta < 0$, $\Delta h = z_{(41)}\left(\text{for } V_t = 0.9V_{\ell(31)}\right)$.

Table 10

Summary of Supersquat Calculations at 1:2 Transition

For 14-ft Depths and Various Widths

Variable	14' × Base		14' × 200'		14' × 250'	
	Section Numbers					
	31	41	31	41	31	41
A	4955.0	2772.1	4955.0	3487.1	4955.0	4202.1
W	396.5	242.1	396.5	292.1	396.5	342.1
$n = A/a$*	5.552	3.106	5.552	3.907	5.552	4.708
$\overline{D} = A/W$	12.497	11.450	12.497	11.938	12.497	12.283
$\sqrt{g\overline{D}}$	20.060	19.201	20.060	19.606	20.060	19.887
u_ℓ	5.892	6.844	5.892	6.525	5.892	6.208
z_ℓ	2.374	2.134	2.374	2.291	2.374	2.360
V_ℓ	10.028	6.619	10.028	8.044	10.028	9.138
$0.9V_\ell$	9.025	5.957	9.025	7.240	9.025	8.224
$z_{0.9V_\ell}$	1.093	1.053	1.093	1.102	1.093	1.110
$u_{0.9V_\ell}$	3.297	4.208	3.297	3.869	3.297	3.571
$0.9V_{\ell_{31}} - V_{\ell_{41}}$	2.406		0.981		-0.113	
η**	1.435		0.597		0.00	
$\Delta h = \eta + z_{\ell_{41}}$ †	3.569		2.888		1.861	
$1 - \frac{\Delta h_i}{\Delta h_1}$	-33.36		-7.93		30.47	

* a = design tow cross-sectional area = 8.5 ft × 105 ft = 892.5 sq ft.

** $\eta = \overline{D}_{41} / \sqrt{g\overline{D}}_{41} \left(0.9V_{\ell_{31}} - V_{\ell_{41}}\right)$.

† If $\eta < 0$, $\Delta h = z_{41}$ (for $V_t = 0.9V_{\ell_{31}}$).

Table 11

Summary of Tow Clearance Calculations at Various Canal Conditions

Canal Size	Design Clearance Under 8-1/2-ft Tow ft	Supersquat Δh ft	Supersquat Clearance ft	Net Design Clearance* ft
Base × Base	0.5	2.7	Grounding	Grounding
Note: Increasing canal width without depth increase does not prevent grounding at steady or supersquat tow speeds.				
12' × Base	3.5	3.2	0.3	Possible grounding
12' × 200'	3.5	2.6	0.9	Possible grounding
12' × 250'	3.5	2.1	1.4	Marginal
12' × 300'	3.5	1.0	2.5	1.0
13' × Base	4.5	3.4**	1.1	Marginal
13' × 200'	4.5	2.7**	1.8	0.3
14' × Base	5.5	3.6	1.9	0.4
14' × 200'	5.5	2.9	2.6	1.1
14' × 250'	5.5	1.9	3.6	2.1
15' × Base	6.5	3.8	2.7	1.2
15' × 200'	6.5	3.0	3.5	2.0
15' × 250'	6.5	1.8	4.7	3.2
15' × 300'	6.5	1.2	5.3	3.8

* Supersquat clearance minus lock surge allowance (1.0 ft) and minimum bottom clearance (0.5 ft).

** Linearly interpolated supersquat.

APPENDIX A: NOTATION

a	Tow cross-sectional or frontal area, sq ft
a_1	Surge amplitude for the present unmodified canal, ft
a_n	Surge amplitude for any given variable canal geometry, ft
A	Cross-sectional flow area of channel, sq ft
C	Celerity of surge in a channel in ft/sec; for small amplitude surges, $C = \sqrt{g\bar{D}}$
$\bar{D}$	Mean depth of channel, ft, $\equiv A/W$
$\bar{D}_1$	A_1/W_1 present canal mean depth, ft
$\bar{D}_n$	Variable geometry canal mean depth, ft
g	Acceleration due to gravity, ft/sec^2
h	Water-surface elevation above mean sea level, ft
n	Manning's resistance coefficient for surge computations; ratio of channel area to tow area for tow squat computations
q	Lateral inflow per unit distance along the channel per unit time, sq ft/sec
Q	Total channel discharge, cu ft/sec, normally $Q = AV$
R	Hydraulic radius of channel, ft
u	Return velocity, ft/sec
u_1	Initial estimate of return velocity for steady squat computations, ft/sec
u_ℓ	Return velocity at limiting tow speed, ft/sec
u_{k+1}	$k+1^{th}$ estimate by iteration
V	Mean flow velocity of channel, ft/sec
V_ℓ	Schijf limiting tow speed, ft/sec
$V_{\ell (2)}$	Schijf limiting velocity at given channel cross section (2), ft/sec

V_t	Tow speed, ft/sec
$V_{t_{(1)}}$	Tow speed at given cross section (1) for calculation tow supersquat, ft/sec
W	Top width of water surface of channel, ft
W_1	Present canal top width, ft
W_n	Variable geometry canal top width, ft
z	Vertical tow squat, ft
z_1	Initial estimate of tow squat for steady squat computations, ft
z_{k+1}	$k+1^{th}$ estimate by iteration
z_ℓ	Tow squat at limiting tow speed, ft
Δh	Magnitude of tow supersquat at an abrupt channel transition, ft; $\equiv z_\ell + \eta$
Δt	Time-step for lock surge model computations, sec
Δx	Space-step for lock surge model computations, ft
η	Positive surge magnitude induced by a tow driving through an abrupt channel transition, ft
$\partial/\partial t$	Rate of change with respect to time, per time
$\partial/\partial x$	Rate of change with respect to distance, per ft
$0.9V_{\ell_{(1)}}$	90 percent of limiting tow speed at cross section (1) , which is the normal upper possible speed of self-propelled tows, ft/sec

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Huval, Carl J
  Lock approach canal surge and tow squat at Lock and Dam 17, Arkansas River project; mathematical model investigation / by Carl J. Huval. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1980.
  53, [12], 2 p. : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; HL-80-17)
  Prepared for U. S. Army Engineer District, Tulsa, Tulsa, Oklahoma.
  References: p. 53.

1. Arkansas River. 2. Canalization. 3. Lock filling and emptying systems. 4. Locks (Waterways). 5. Mathematical models. 6. Navigation channels. 7. Squat of vessels. 8. Surges. I. United States. Army. Corps of Engineers. Tulsa District. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; HL-80-17.
TA7.W34 no.HL-80-17